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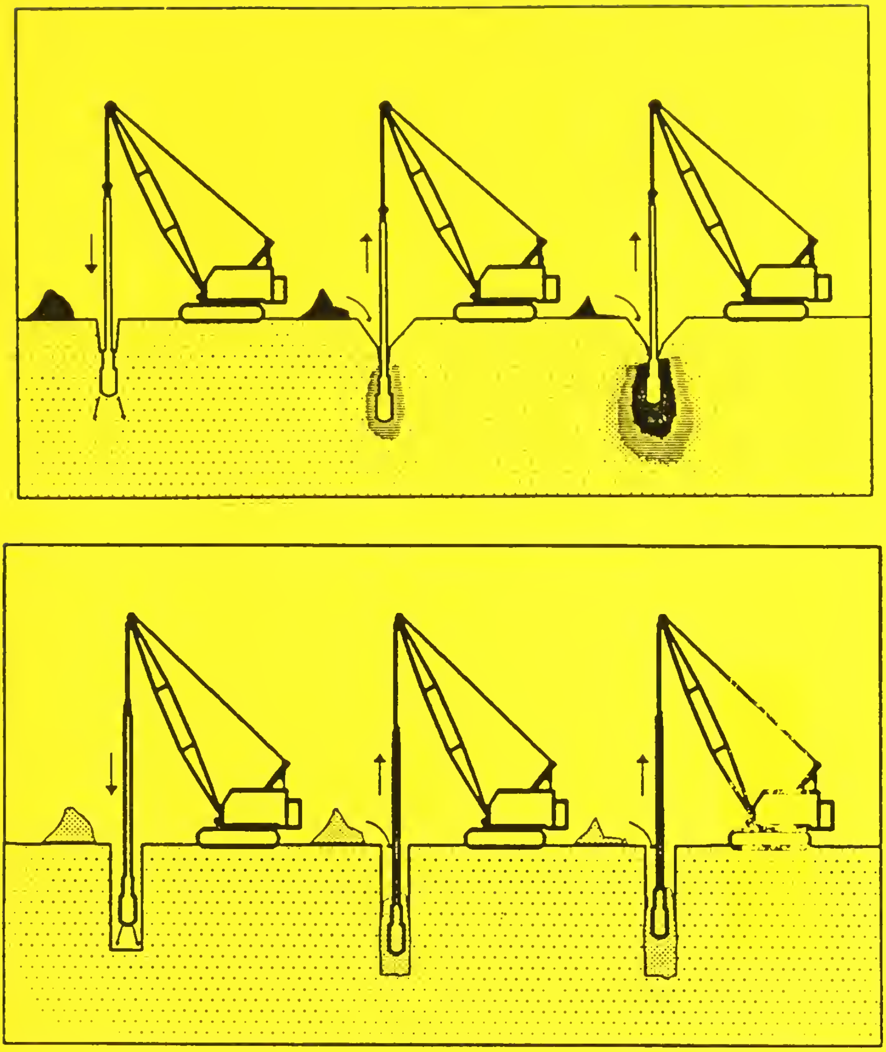

Soil Improvement Through Vibro-compaction and Vibro-replacement.

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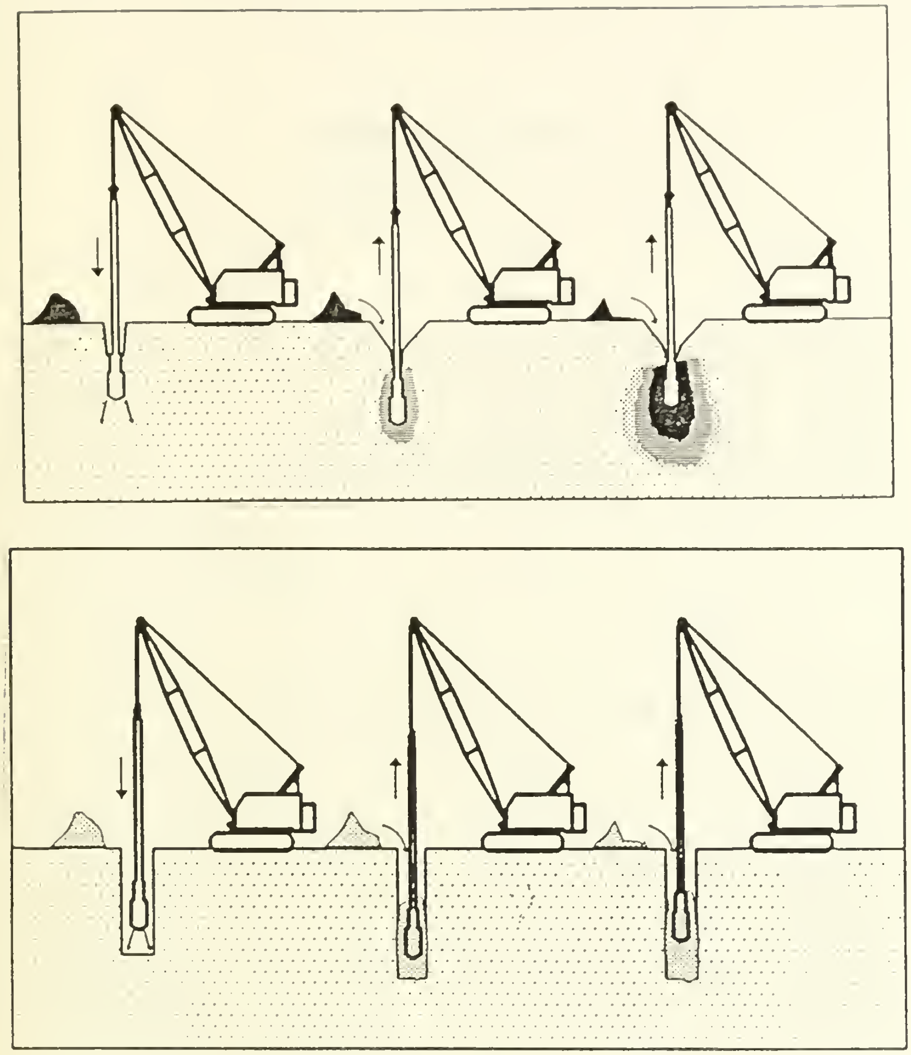
Dept. of Civil Engineering

28 June, 1991



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Synopsis

In recent years, a great deal of emphasis has been placed on the various techniques of ground stabilization, or ground modification as it will be termed henceforth. For the most part, these techniques are not new, with their development having evolved through the first half of this century. However, in the past most of these techniques were looked at as a desperate means, or last attempt at stabilizing unsuitable soil conditions. There was an air of mystic surrounding these techniques. Most early material on the subject (as well as a large extent of recent publications) was produced by contractors specializing in this field. Unsurprisingly, most of the information available on the techniques, focus on the success obtained by their methods. Therefore, the failures, or lack of successful application (as termed by one contractor), have gone unrecorded. Through the years, leading up to todays construction environment, these ground modification techniques have undergone refinement. There has been a maturing of the processes, and a slow, but consistent integration of these techniques with todays normally planned construction process.

The reasons for this turn around are two-fold: With todays highly integrated and often inflexible construction programs, delays in below-ground work (often the first major phase of construction after site clearance) could disrupt the interlocking follow-on construction and result in lengthy and more important in todays industry, costly delays. Experience, as well as logical thinking has dictated that it is preferable to anticipate "unforeseen" ground conditions and integrate the solutions into the original schedule (be it critical path method, bar chart, critical task, etc.). The alternative to prior planning are delays when difficult unanticipated problems are encountered. In todays construction industry liability claims and surety oversight have made the "quick fix" field change a thing of the past. In addition, the methods of ground modification described herein, although not extremely complex in nature, are not techniques that can be ordered over the phone with delivery and correction of unforeseen conditions occurring the next day.

An additional factor in bringing ground modification techniques to the forefront are environmental concerns regarding depleting natural resources. Awareness of the growing need for conservation and less waste has prompted the industry to re-think the site selection of projects. Sites that were previously thought of as unsuitable and were disregarded are being developed out of necessity. Land is being reclaimed through the use of ground modification techniques.

The problems due to unstable ground dealt within this paper are those normally associated with dangerous or excessive settlements on soft or loose soils, as well as the strength requirements of soft clays. Two major ground modification techniques will be examined: Vibro-compaction and Vibro-replacement.

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I. Vibrodensification

A. Introduction

Of all the different construction materials, methods, techniques, and processes, one common bond remains among all projects undertaken. The majority of all of man's construction has been done on, in, or with soil (Mitchell, 1981). With the increase in environmental concerns and availability of suitable construction sites decreasing, the need to utilize what were once thought of as poor soil areas for construction is increasing. With the advent of ground modification techniques (past, present, and future) the use of natures most abundant building material, soil, can be extended.

The basic concept of soil improvement, specifically drainage, densification, and reinforcement were developed hundreds or thousands of years ago and remain valid today (Mitchell, 1981). With the advent of machines in the 19th century, these processes have shown great increases in the quantity and quality of work completed. Probably one of the most significant improvements has been the introduction of the vibratory techniques used to densify soils.

Prior to discussing the specific techniques of vibrodensification, a brief definition of ground modification is necessary. The term ground modification has been developed by GKN Hayward-Baker to describe the specialty that encompasses the full range of techniques now available to densify or otherwise improve the ground as an integral part of the construction system. In short, ground modification is the in-place controlled improvement of ground materials to form part of the geotechnical construction system (Welsh, 1991). Some of the technologies include vibrodensification (vibro-compaction and vibro-replacement), dynamic compaction, chemical, jet and compaction grouting, slurry trench cut-off walls, mini piles, tiebacks, lime injection, and ground freezing, to name a few. Each of the above techniques, although mature in theory, are just reaching adolescence in practice. Each have undergone significant improvements in the past twenty years, in addition, each could be and has been discussed in papers devoted to entirely one subject. For this reason, only vibrodensification techniques will be discussed, namely vibro-compaction and vibro-replacement (stone columns).

B. Vibro-Techniques

In the mid 1930's, the use of in place vibrators to densify soil was patented in Germany. Although evidence of the first sand pile usage points to the French Military Engineers in the 19th century, the modern origins truly began in Germany. Russian emigre Sergei Stevermann and Wilhelm Degen had an idea for compacting cohesionless soils both above and below the water table (Glover, 1982). Both agreed the best method would achieve effective compaction only when the vibrator was placed into the soil at the location the compaction was required. The vibratory equipment would have to be in direct contact with the soil while emitting its horizontal vibratory forces.

It is reported that the political atmosphere in Germany during the late

1930's forced Stevermann to leave the country and migrate to the United States. Degen remained in Germany and produced the first working vibratory machine in 1936-37 (Glover, 1982). Stevermann produced his own machine soon after and expanded the process with the formation of the Vibroflotation Foundation Company of Pittsburgh.

Because of their earlier work together, both Steverman and Degen developed their machines on a similar theory. Both believed that vibrations of an appropriate form could eliminate the interangular friction of cohesionless soils so that those that were initially loose could flow by gravity into a dense state (Bell, 1975). A poker vibrator was developed that hung vertically from a crane boom. This allowed the poker to penetrate to depths greater than those obtained by surface compaction. The poker, which is now known as a Vibroflot (Figure 1), can also operate efficiently below ground-water thus compacting soils normally inaccessible without drainage.

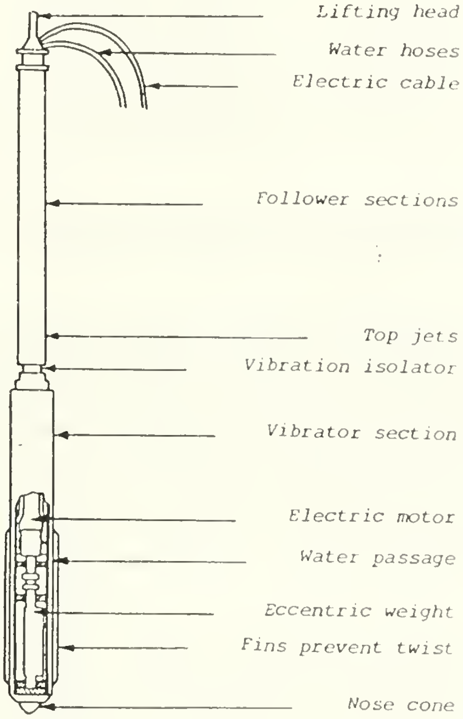

Lifting head
Water hoses
Electric cable
Follower sections
Top jets
Vibration isolator
Vibrator section
Electric motor
Water passage
Eccentric weight
Fins prevent twist
Nose cone

Figure 1. Essential features of the Vibroflot. (Glover, 1982)

Prior to examining the two methods of soil reinforcement, it is important to understand some basic concepts of soil densification. The mechanical improvement of soil can be carried out in two ways. In the case of permeable soils, densification can be implemented (i.e., Vibro-compaction), and in the case of soft or low permeable soils, reinforcement (i.e., stone columns) is used.

The densification of soils in-situ and their reinforcement by stone columns are not competing processes, but complimentary (Wallays, 1982). Stone columns or reinforcement is used when the soil cannot be densified, which in-turn leads to the process of densification.

Soil densification in simple terms is the increase of density with the decrease of volume occupied by the voids (Wallays, 1982). This process is achieved through the introduction of additional material in the constant volume, or the decrease of total volume occupied by in-situ material. Figure 2 shows this process with uniform spherical particles. The amount of material (i.e. particles) remains constant, however, the area occupied decreases.

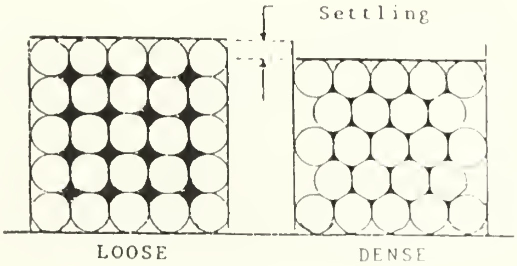


Figure 2. Soil densification and rearrangement after compaction. (Besancon, 1982)

For material to shift from a loose state to a dense state, two conditions must be satisfied (Wallays, 1982). First the individual particles must slide over one another. That is, the shear resistance force at the points of contact must be overcome. The shear resistance force is a function of the normal force, coefficient of physical friction, and the adhesion force. Secondly, when the soils are located below the ground water level, or the addition of water will occur (i.e., jetting during vibroflot penetration), the pore water pressure corresponding to the reduction of voids must be able to freely and quickly dissipate.

In order to start the sliding process, and thus the densification process, the force applied to the soil particles must be greater than the interparticle shear resistance force. For this reason, it is possible to reach an upper particle size limit that is suitable for Vibro-compaction. Extremely large particles (i.e., rock fills) have shear resistance forces that cannot be overcome with economically feasible vibroflotation equipment.

The dissipation of pore water pressure, which for saturated soils is caused by a decrease in void volume, requires that the in-situ material have a large enough permeability to allow the excess pore water created during densification to flow freely.

> When the soil permeability is low, the pore water becomes momentarily pressurized, but practically none of the pore water volume flows away. When the soil is plastified, soil displacement occurs practically at constant volume, i.e. without any actual densification. When the soil permeability is intermediate, the excess pore water pressures generated during the shock, which decrease from the point of application, can cause some drainage of the pore water, so that a partial decrease of volume of voids occurs. The soil displacement results in this case, first from the densification corresponding to the volume of drained water, and second, from the additional displacement at constant volume. The larger the soil permeability, the larger the densification. (Wallay, 1982).

Permeable materials have a maximum and minimum dry unit weight. Regardless of the technique used, it is not possible to increase the unit weight of a dry material above the maximum dry unit weight. Conversely, it is not possible to decrease the unit weight of a dry material below the minimum dry unit weight. To obtain maximum and minimum dry unit weights refer to ASTM D4253 and D4254. It is important to remember that the unit weights of soils are dependent on grain size distribution, the shape, and angularity of the particles (Wallay, 1982).

Although simple in theory, the design engineer should keep in mind the densification of a material depends on the initial value of the dry unit weight (i.e., dry unit weight in-situ). If the in-situ dry unit weight is close to the minimum dry unit weight obtainable, a large amount of densification is possible. Conversely, if the dry unit weight is relatively high to start, a large increase can hardly be expected.

To check the densification achieved, one of three methods can be used (Wallay, 1982):

- The measurement of relative density
- The blow count in the SPT tests (i.e., "N" value)
- The cone resistance measured in the CPT test.

The specifics of the above methods will be discussed in the corresponding applicable sections.

C. Vibroflot

The vibroflot is the common link between vibrodensification techniques. The vibroflot is used in both Vibro-compaction and Vibro-replacement techniques. Figure 3a depicts a schematic drawing of the essential features of the vibroflot. Figure 3b depicts the vibroflot in the field attached to a crawler crane.

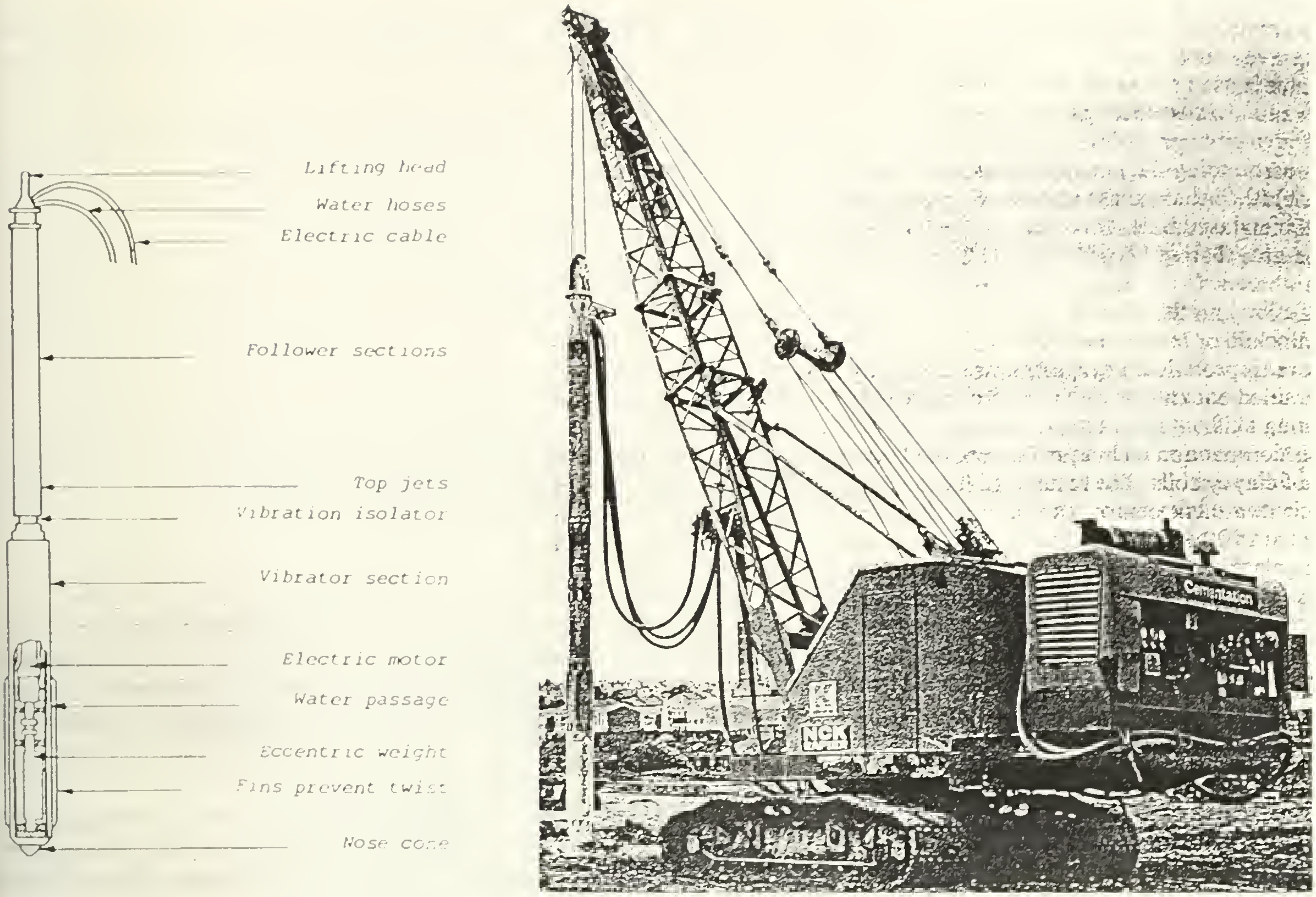

Lifting head
Water hoses
Electric cable
Follower sections
Top jets
Vibration isolator
Vibrator section
Electric motor
Water passage
Eccentric weight
Fins prevent twist
Nose cone
Cementation
NCK

Figure 3. a.) Schematic drawing of the Vibroflot
b.) Field application of the Vibroflot.
(Bell, 1975)

The vibroflot is essentially a long slender steel tube with two parts, the vibrator and the follow up tubes. An essential feature of the vibroflot is its laterally vibrating element at its bottom. The vibrator (the heart of the vibroflot), consists of a 300-400 millimeter diameter, hollow cylindrical body, 2.0-4.5 meters in length. The vibrator is connected to the follower tubes by an elastic coupling or "universal" type joint to isolate it. Eccentric weights in the lower part of the vibroflot are driven by an electric or hydraulic motor.

In the early stages of development, it was soon discovered that a simple vibrator range limited the range of compactible soils. In addition, it was economically impractical to extend this range by complex machines with vibration parameters adjustable to in-situ resonant frequencies. However, most recent developments have been aimed at matching machine characteristics

more closely with soil properties by providing alternating frequencies, amplitudes, and power levels within a single machine (Bell, 1975).

Recent developments have also been made in casing drivers, pile drivers, and vibratory hammers, which have lead to these being classified as vibroflots. This is an incorrect term for these machines. The apparatus mentioned above include a top mounted motor, which produces an axial (vertical) vibration in the length of a continuous tube. This type of set-up produces excellent results in penetration of frictional soils, but does not compact in radial directions, thus if used for vibrodensification techniques, the centers must be very closely spaced.

Recent developments have also seen the tendency toward hydraulic powered vibroflots replacing electrically powered vibroflots. The use of hydraulics allows the generation of greater power from a motor of a relatively small volume. By doing this, the dimension of the machine that affects penetration can be kept small.

The eccentric weights in the lower part of the vibroflot operate at 1800 revolutions per minute in a horizontal plane. Up to 34 tons of centrifugal force can be generated, creating amplitudes as great as 25 millimeters at the tip of the vibroflot (Glover, 1982). The most common operating frequencies are 30 Hz and 50 Hz (Mitchell, 1981).

The total weight of the vibroflot is adjustable by the addition of heavy or light-weight follower tubes. When added up, these can produce from 4 to 8 tons per 12 meter long vibroflot. The follower tubes on which the vibrator is suspended, may also be added in sections so that any reasonable desired depth of treatment can be achieved. Normally, treatment depths of over 8 meters are not required, however, depths exceeding 30 meters have been recorded (Mitchell, 1981).

Vibroflot sinking rates of 1-2 meters/minute and withdraw/compaction rates of about 0.3 meters/minute are typical (Mitchell, 1981). In addition, water pressures of up to 0.8 megapascals and flow rates up to 3,000 liters/minute may be used to facilitate penetration. The zone of improved soil ranges from 1.5 meters to 6.0 meters from the point of penetration, depending on the in-situ soil properties.

1. Basic Operating Technique and the Role of Water

The vibroflot is used for both Vibro-compaction and Vibro-replacement, that is, for both the compaction of cohesionless soils and for the formation of stone columns. The basic techniques are virtually the same for both with minor variations according to soil type and usually occurring during withdraw of the vibroflot.

As mentioned previously, the vibroflot is usually suspended from a

crawler crane. When penetrating into the ground, it usually relies on its own weight. However, water and air are often employed to assist. Jets of water and/or air can be activated from the lower conical point. Although usually not essential to vibroflot penetration, these fluids prime function are to support the borehole during treatment.

The water jets at the tip are employed whenever the borehole formed by the vibroflot is likely to be unstable, and always when the possibility of ground water infiltration is present. The use of water creates an annulus space about 50-100 millimeters surrounding the machine (Bell, 1975). The circulation of the excess water is encouraged to overflow the borehole. This process relieves excess hydrostatic pressure and outward seepage forces help stabilize the uncased hole. The upward water flow in the hole also helps to remove the smaller silt-size particles, forming a cleaner compacted area. If the vibroflot is being used for Vibro-compaction, care must be taken to reduce the water flowing upward to allow for the sand-sized backfill or in-situ sand to fall into place. This requires considerable operator skill. This technique is equally effective for Vibro-replacement techniques in clayey soils.

Occasionally it is necessary to use a dry technique when forming stone columns. This is especially true in city-center sites, where disposal of waste water (including suspended solids) can be a problem. In addition, the dry technique is suited for isolated areas or developing countries, where large quantities of water may be scarce. In this method, the vibroflot penetrates the soil by shearing and displacement, thus the term Vibro-displacement is sometimes used. Since jetting water is not used, no annulus between the machine and the bore is formed. Because of this, the vibroflot must be removed prior to the addition of granular material. The bore hole can also create a suction causing the collapse of the uncased hole. For this reason, compressed air is circulated through the conical tip to ease the withdraw process. Great care should be taken when using compressed air. The combination of standing water and compressed air could result in a soft slurry forming inside the hole, resulting in an unsound column. In addition, weak soils with shear strength less than 20 kiloNewtons per meter squared (Mitchell, 1975) pose a risk that poorly regulated air flow will fissure the surrounding soil. This can be very damaging to the in-situ soil properties.

With the above two methods mentioned, there are clear distinctions between wet (Vibro-replacement) and dry (Vibro-displacement) techniques. In addition, there are clear cut situations where each should be employed.

D. Vibro-Compaction

Vibro-compaction is a vibrodensification technique using in-situ material or borrow material with very similar characteristics as the material in-situ, at the construction site. The process is somewhat similar to concrete vibration by means of a concrete needle vibrator, the aim of each

operation is to increase the compactness. In the field of soil mechanics, a deep vibration treatment, such as Vibro-compaction, results in an improvement of the geotechnical characteristics important in foundation engineering. These characteristics are as follows (Besancon, 1982):

- In-situ density
- Angle of internal friction
- Elastic modulus

By improving the above soil parameters, it is possible to increase considerably the bearing capacity and to reduce the settlements under structural loads.

The deep vibro process achieves cylinders of compacted soil. However, the effectiveness of the technique depends on the in-situ soil characteristics. The results of Vibro-compaction are not identical for every soil encountered. Granular soils display very different results then those of cohesive soils.

1. Process

Vibro-compaction is the term coined for the treatment performed on non-cohesive granular soils. The technique behind Vibro-compaction is quite simple. It relies on the fact the intergranular forces between cohesionless soils can be overcome by the effects of vibration. The rearrangement of the soil grains under the action of gravity achieves a maximum compactness. This same principle is used when determining maximum densities in relative density tests. Figure 4 shows an oversimplified case of the effects of Vibro-compaction on soil particles.

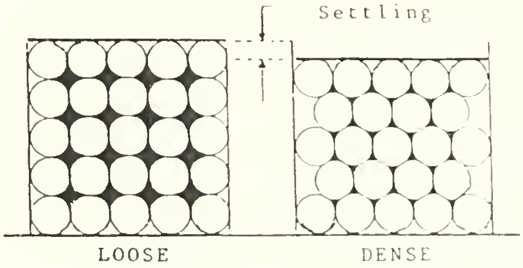


Figure 4. Simplified case of soil particles subjected to Vibro-compaction. (Besancon, 1982)

As can be seen in the figure, the void ratio in the layer that is subject to vibrations decreases. This decrease in void ratio induces settlements in the layers above. This process occurs repeatedly until the vibroflot reaches the surface.

It was discovered early on, that shear failures were unlikely to occur with normal foundation loadings on loose granular soils, but the consolidation settlements could be excessive for certain types of structures (Bell, 1975). If the loads were from machinery, that is, transmitting vibrations into the soil, consolidation settlements were increased. The use of Vibro-compaction can be thought of as a pre-load, per say, for machinery foundations through the process of vibro-compaction. The in-situ soil is subjected to vibrating conditions forcing the rearrangement of particles and subsequent ground subsidence.

2. Material

Vibro-compaction techniques are best suited for densification of clean, cohesionless soils. Experience has shown that they are generally ineffective when the percentage by weight of fines (particles finer than No. 200 sieve or 0.074mm diameter) exceeds 25% (Mitchell, 1981). The ineffectiveness in this situation is due to decreased impermeability of material with excessive fines. It will not allow the rapid drainage of pore water pressure required for densification after liquefication under the vibratory forces. It is also likely the increased intergranular forces of the cohesive materials are more difficult to disrupt. However, it has been reported (Mitchell, 1968) that good success has been reported in soils containing over 30% fines by weight.

Figure 5 shows a breakdown of the most desirable size range for soils densified by Vibro-compaction according to U.S. Navy Standards.

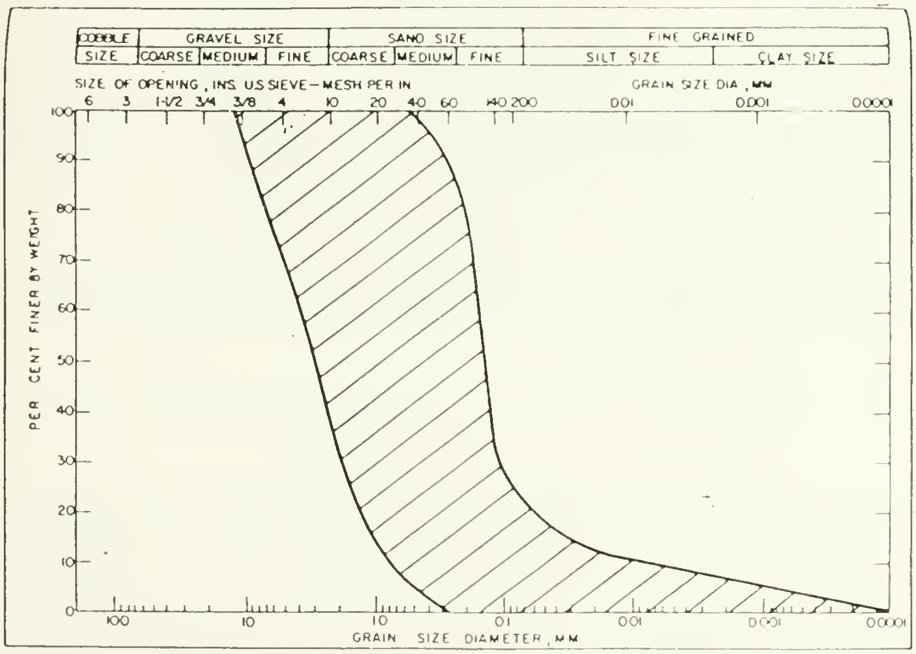


Figure 5. Desirable size range for soils densified by Vibro-compaction. (NAVFAC DM 7.3)

Figure 6 depicts the range of soils found to be most liquefiable (Lee and Fitton, 1968).

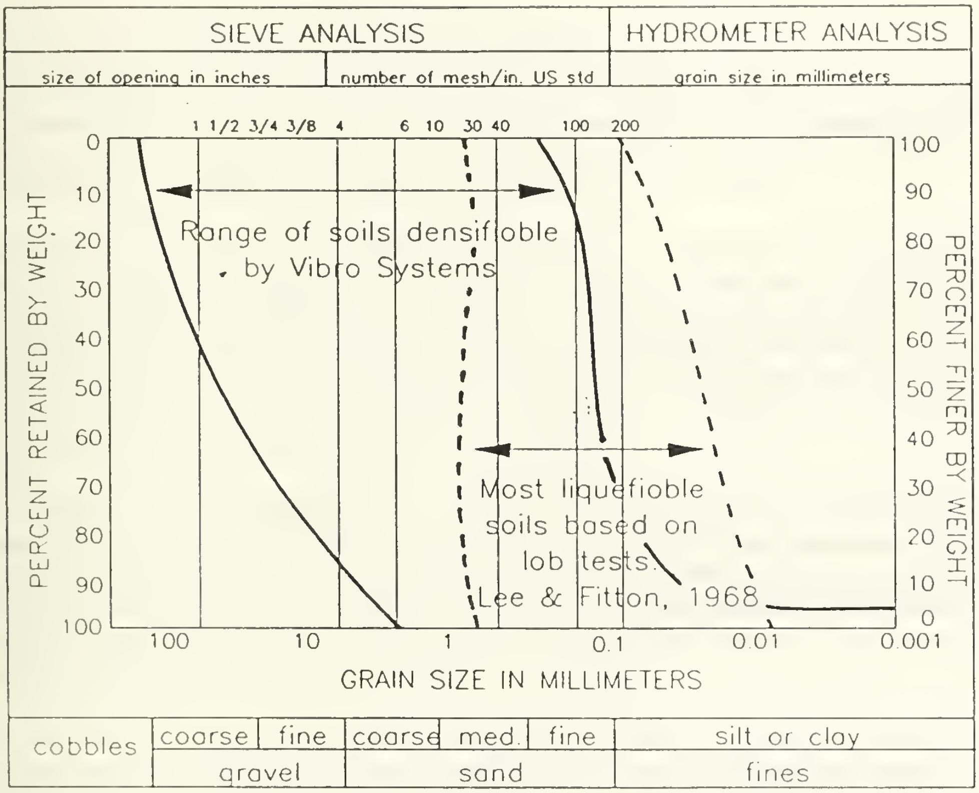


Figure 6. Grain size distribution of the most liquefiable soils. (Hayward-Baker, 1988)

As shown in Figure 6, the forces involved in liquefication are similar to those induced during Vibro-compaction and therefore the grain size distributions overlap.

As stated previously, the essential piece of equipment in the Vibro-compaction and Vibro-replacement method is the vibroflot. To begin the process, the vibroflot and supporting equipment is fastened to an overhead crawler crane boom. The vibroflot is positioned over the selected point to

receive the Vibro-compaction technique. The process begins by lowering the vibroflot into the soil to the desired depth; when used within the designed range of material suited for Vibro-compaction the vibroflot will reach the desired depth under the weight and vibratory action of the vibroflot itself. The total weight of the vibroflot is roughly 2 tons (dependent on the exact equipment manufacturer). The vibroflot will typically develop a sinking rate of between 3 feet and 6 feet per minute. Once the desired depth is obtained, the water jetting is shifted from the nose of the vibroflot to the top of the vibroflot. The volume of water is also adjusted to allow for the densification of the in-situ particles.

During the compaction stage (withdrawal) the vibroflot is raised slowly enough to produce the needed densification; typically a rate of one foot per minute is appropriate. When used in clean, coarse sands, an increase in density causes the resistance to vibroflot motion to increase, thus increasing the motor energy. The energy increase, when monitored, can provide the basis for controlling the compaction process (Mitchell, 1968).

Additional backfill soil supplied from the surface is also compacted to a high density from this process. Backfill used can either be from the site or borrow of similar soil characteristics of the in-situ material. When clean, free draining soils are subjected to Vibro-compaction, 3 cubic feet to 20 cubic feet of material may be required per foot of compacted depth, and a cylindrical column 8 feet to 10 feet in diameter is compacted by one penetration of the vibroflot (Mitchell, 1968).

When computing the amount of densification by monitoring the amount of settlement of fill used, it is important to account for some of the original in-situ soil washing out during vibroflot penetration. The degree of compaction is maximum at the center of the cylindrical column, and decreases with radial distance. The amount of compaction is proportional to the amount of vibration energy transmitted radially outward.

The radius of influence decreases from about 6 feet for clean sands to 2 or 3 feet in sands containing more than 25% fines (Mitchell, 1981). Depths of greater than 100 feet have been compacted successfully by Vibro-compaction (Welsh, 1991).

3. Design

Common among all foundation design problems is establishment of the distribution of contact stresses anticipated. Working in conjunction with a structural engineer, the geotechnical engineer will choose the desirable foundation sizes and depths based on anticipated settlements within a tolerable limit. All foundations, therefore, should be designed to provide support with minimum differential settlements. When pushed, most structural engineers will agree the majority of structures can withstand 0.25 inch differential settlements and remain unaffected, due largely to methods used in

the design of the structure members (Aggour, 1991). Therefore, the exploitation of tolerance differentials on vibro-compacted soils can often permit the use of low-cost foundations with great economy (Mitchell, 1981).

The design of Vibro-compaction techniques breaks down to two factors - depth and spacing. Depth of the treatment can usually be determined by the correlation of induced stresses by the anticipated foundation loads. Spacing is determined by the degree of improvement of the soil properties required to limit settlements and to achieve safe bearing capacities.

4. Depth of Treatment

When using conventional pilings, the soil properties take on a secondary importance as compared to the characteristics of the piling itself (especially true in end-bearing designs). Unlike conventional piling, Vibro-compaction improves the existing soils, and therefore relies on the in-situ soil properties for support of structures. This improvement often allows the use of conventional spread footings at relatively shallow depths.

When defining the depth of treatment for Vibro-compaction, conventional stress theory should be applied. It is often safe to assume elastic stress distributions patterns apply. This theory is best exemplified when designing foundations of a comparatively small area. Following elastic theory principles, the significant influence of the stress bulb on surrounding soils may not affect soils at depths greater than twice the width of the foundation. When designing narrow foundations, treatment depths rarely exceed three times the footing width (Bell, 1975). For wider foundations, the necessary depths may be half the width of a raft foundation. This is due to the reduction in compressibility of frictional soils with the increase in overburden pressure; therefore it is rarely necessary to compact these to depths greater than 25 feet (Mitchell, 1981).

Due to the relative shallowness of the treatment depths, the area of treated soils will often lie entirely within a homogeneous stratum layer and never penetrate into underlying stronger materials. This is caused by the densification and strengthening of the overlying homogeneous layer.

When boring logs indicate a soil layer strong enough to support anticipated loads lies within the significant stress bulb (i.e., at depths less than twice the width of the foundation), it is not necessary to treat depths beyond this layer. Allowing the vibro-compacted areas to penetrate approximately 3 feet into the stronger underlying soil layers will adequately transfer the loads to the stronger incompressible underlying layers.

When designing foundations for vibrating machinery or to withstand substantial earthquake damage, more complex factors will determine the depth of treatment. In these cases, the process of Vibro-compaction acts to subject the soil to greater dynamic stresses than those anticipated from subsequent

shocks (Bell, 1975). When applied in this manner, Vibro-compaction reduces the risks of further settlement or liquefication. The design of dynamic loads and the effect on vibrodensification is beyond the scope of this paper.

5. Spacing

The spacing of vibro-compacted areas will ultimately determine the properties of the soil area on which the foundations will be placed. The problem facing engineers is to provide adequate incompressibility and strength at all locations between treated columns while minimizing the number or areas vibro-compacted. That is to say, provide an adequate strengthened soil with maximum spacing, thus reducing the overall costs.

The spacing of vibro-compacted columns is mainly dependent on the ability of the soil to densify under the vibratory action of the vibroflot. The soils properties in turn dictate the radial distance in which soil particles will be affected (i.e., densified) under vibratory action. Without discussing complex soil mechanics theories, it is basically cohesion and permeability that affect densification of soils. Cohesion is the interparticle forces found in silts and clays, while permeability is the ability of water to flow through soils. Cohesion is predominantly in silts and clay, and generally increases as the percentage of fines (clay particles) increases. Permeability is also affected by the amount of fines present. Clean granular material has larger voids, thus allowing for a greater amount of water flow then cohesive (silts and clays) material. However, if fines (silts and clays) are mixed throughout the granular material, these take the place of the voids, thus reducing the amount of water flow (permeability).

Cohesion will prevent the densification of soils, except by Vibro-compaction. The forces generated by the vibroflot in cohesive soils do not penetrate radially as well as those in granular soils. The effects of Vibro-compaction in cohesive soils are dampened to the extent that the areas of treatment would be closely spaced, making the process uneconomical.

If the in-situ soil has a low permeability, the expulsion of pore water during the relatively short time of vibration is hindered. Soils having a permeability of less than 10 micrometers per second can not be counted on to compact during Vibro-compaction (Bell, 1975). The effect of fines is best shown by examining the increase in penetration resistance by Vibro-compaction in Figure 7.

When compacting soils within the range suitable for Vibro-compaction (i.e., up to 25% fines present by weight) the radius of effective compaction from the point of treatment depends on the specific characteristics of the vibroflot used. With the current machinery, the effective ranges vary between 1.5 meters and 3 meters (Welsh, 1991).

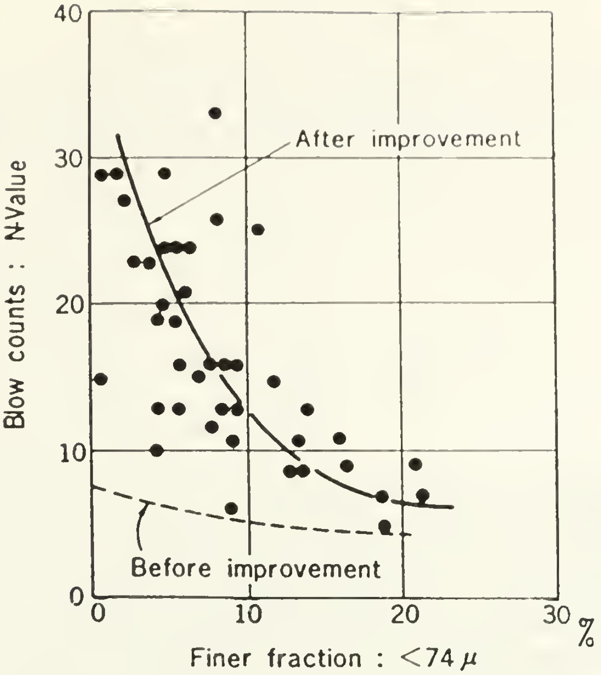
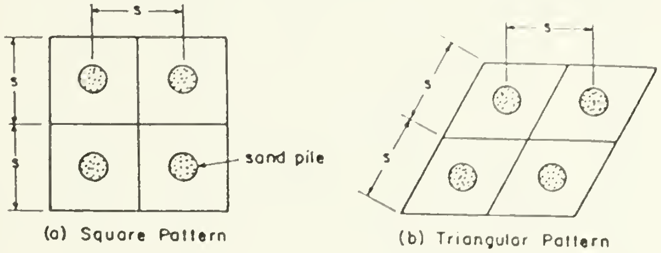


Figure 7. Effect of fines content on penetration resistance by Vibro-compaction. (Saito, 1977)

Vibro-compaction of large areas is done in a grid pattern, either triangular or rectangular with probe spacing usually in the range of 1.5 meters to 3 meters (Mitchell, 1981). Figure 8 shows the two basic types of spacings. This spacing allows overlapping compacted zones covering any desired area. These spacings will provide relative densities on the order of 90% and 60% with an apparent compressibility under strip and pad footings in the range 35-75 MN/m^2 (Bell, 1975).



(a) Square Pattern (b) Triangular Pattern

Figure 8. Usual Vibro-compaction patterns. (Mitchell, 1981)

On sites where large quantities of Vibro-compaction is going to be used, it is wise to run field tests to determine optimum spacing for the most economical compaction desired. However, prior to performing field tests, the following guidelines can be used.

If it is desired to increase the average density of loose sand from an

nitial void ratio e_o to a void ratio e, and if it is assumed that nstallation of a sand pile causes compaction only in the lateral direction, he pile spacings for a square pattern may be determined as follows (Mitchell, 981):

$$S = \left(\frac{\pi(1+e_o)}{(e_o-e)}\right)^{1/2} d$$

nd for a triangular pattern, as follows:

$$S = 1.08\left(\frac{\pi(1+e_o)}{(e_o-e)}\right)^{1/2} d$$

here d = assumed sand pile diameter.

The design approach for a triangular spacing arrangement was first eveloped by D'Appolonia in 1953, and still remains one of the best methods. e determined the radial influence of a single 30 hp vibroflot compaction xtended out about 1.3 m in clean sands (Glover, 1982), the relative ensities achieved at various radial distances from the center of the ibroflot. Figure 9 shows D'Appalonia's work relating these distances to oefficients. Work later done by Brown in 1976 established a curve for a 100 p vibroflot. This curve is also shown on Figure 9. Design curves for the latest equipment (i.e., greater horsepower) could easily be established using imilar techniques.

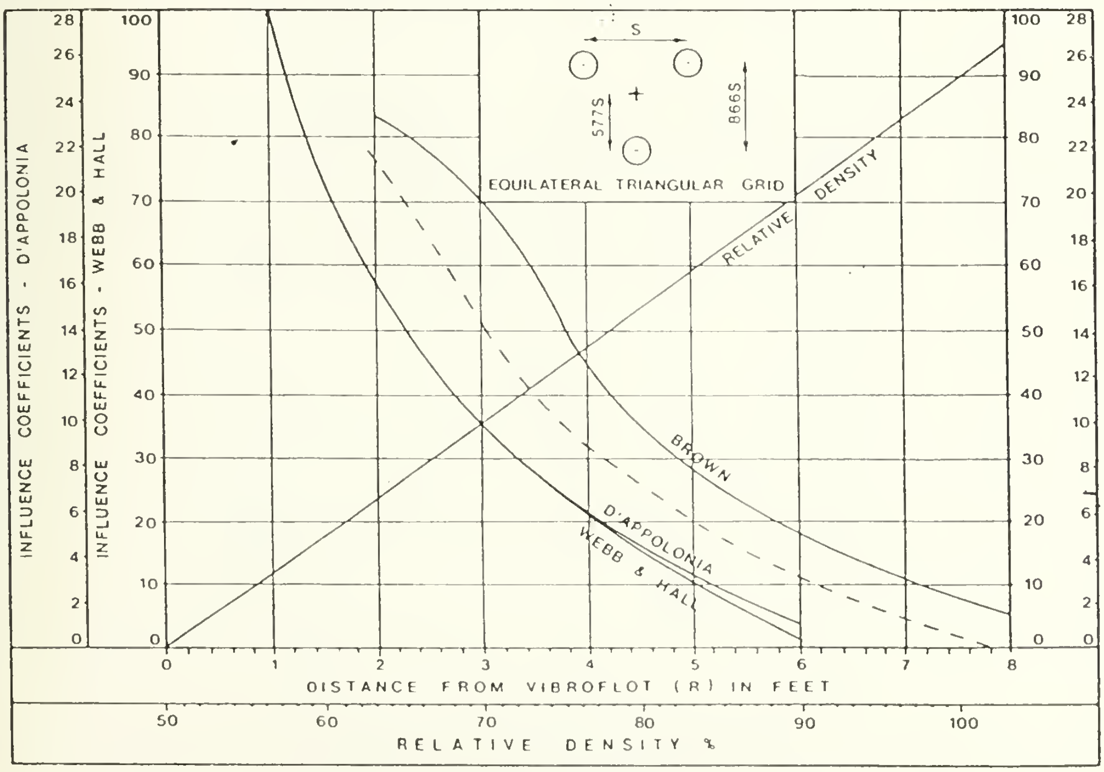


Figure 9. Area pattern design curve. (Glover, 1982)

As discussed earlier, when using Vibro-compaction, it is necessary to supply additional material to achieve the required densities. The fill should be sufficient enough to transmit the vibratory action to the surrounding in-situ material. Coarse granular material with little or no fines provides the best fill material, and allows faster rates of compaction (Glover, 1982). Brown (1976) also developed a rating system for the imported fill material. Brown's system relies on a "stability number" and is shown in Figure 10.

SUITABILITY NUMBER	0 - 10	10 - 20	20 - 30	30 - 50	> 50
RATING	EXCELLENT	GOOD	FAIR	POOR	UNSUITABLE

$$\text{BROWN'S SUITABILITY NUMBER} = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}}$$

WHERE, D_{50}, D_{20} & D_{10} ARE GRAIN SIZES IN MILLIMETRES, AT 50%, 20% & 10% PASSING BY WEIGHT.

Figure 10. Backfill evaluation criteria. (Glover, 1982)

In addition to the empirical formulas, a curve method based on desired relative compaction was developed by S. Thorburn in 1975, as shown in Figure 11.

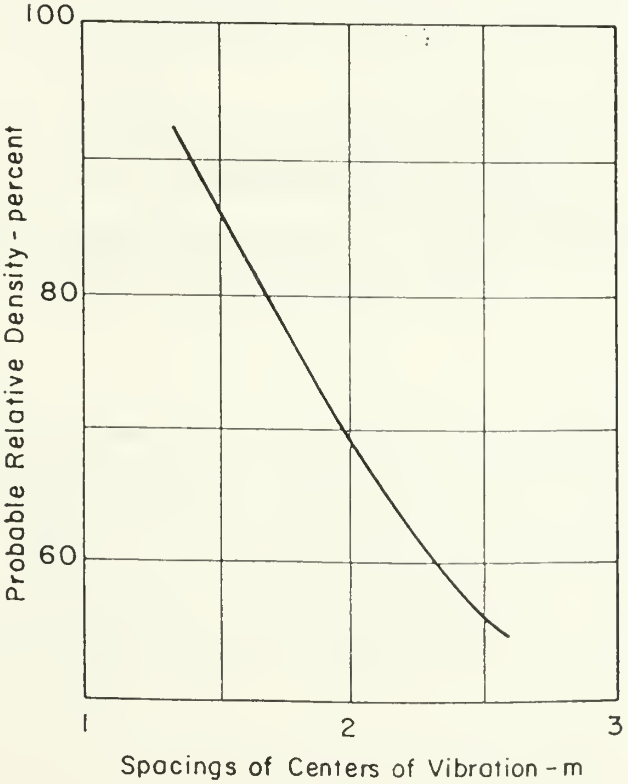


Figure 11. Relative density of clean sand as a function of probe spacing. (Thorburn, 1975)

Design curves relating allowable bearing pressures to limit settlement to 25 millimeters and compaction spacing have also been developed by Thorburn and are shown in Figure 12.

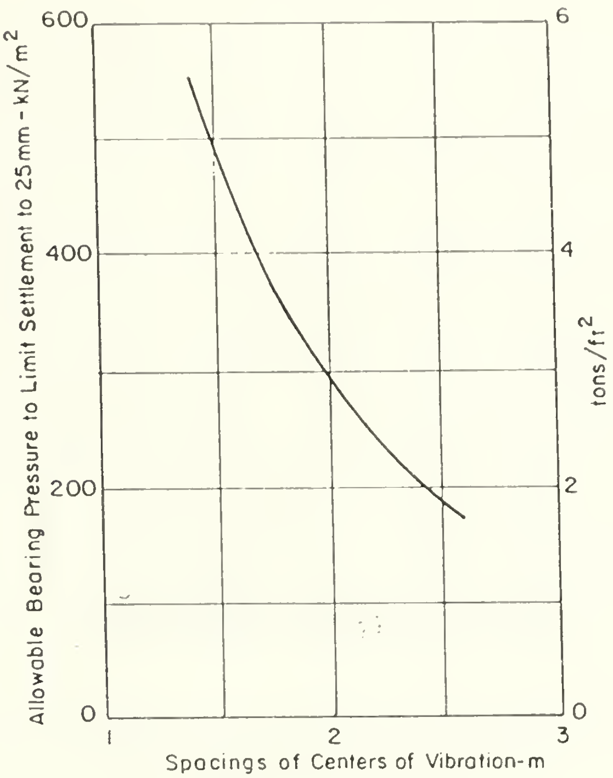


Figure 12. Allowable bearing pressures as a function of probe spacing for footing widths one to three meters.
(Thorburn, 1975)

6. Density Control

The relative increase in soil density at any depth due to Vibro-compaction techniques can be approximated by correlation with cone penetration tests, standard penetration tests, pressuremeter, and other in-situ probes (NAVFAC DM 7.3). These tests, however, must be performed before and after soil treatment. Figure 13 shows the Gibbs & Holtz correlation between relative density and standard penetration resistance. Although this figure is commonly used, it is important to remember the 'N' value measured is also influenced by the effect of vertical stress, stress history, gradation, and other factors (NAVFAV DM 7.3).

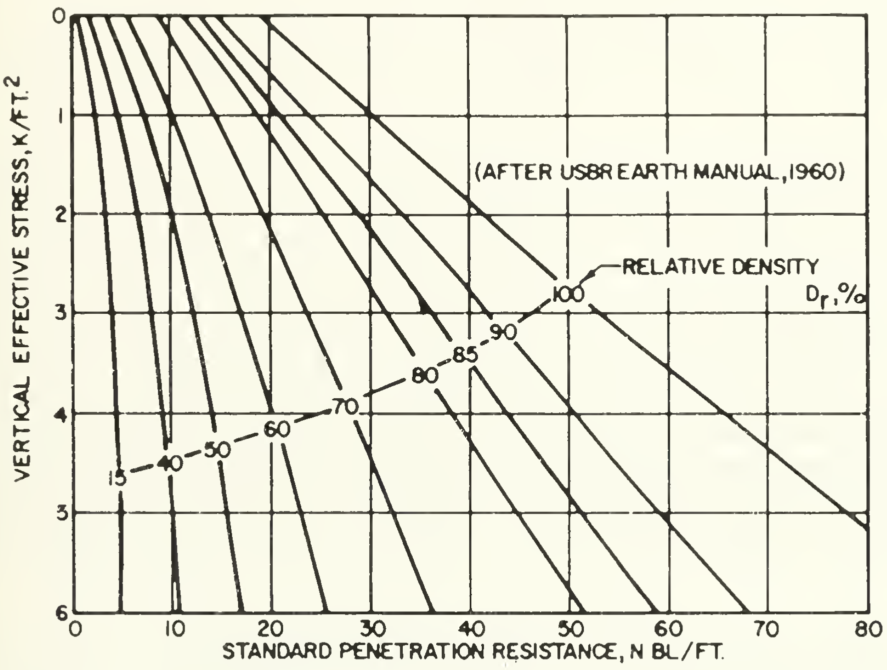


Figure 13. Relative density versus standard penetration resistance. (NAVFAC DM 7.1)

E. Vibro-replacement: Stone Columns

As discussed previously, it is uneconomical to use Vibro-compaction techniques in soils containing greater than 25% fines by weight. When this occurs vibrodensification is achieved by Vibro-replacement. That is, the in-situ soil removed is replaced by granular material, thus giving rise to the name stone columns.

Although stone columns have been documented back to the 1830's (Nayak, 1982), it has not been until recent time (the past 25 years) that stone columns have been re-discovered. Stone columns did not receive acceptance in the United States until the early 1970's (Bachus,1989). Because of the similar techniques involved in Vibro-compaction and stone columns (Vibro-replacement), it is not surprising to find stone column technology originating in Germany with the company Wilhelm Degen founded (Glover, 1982). Stone column technology is a logical branch of vibrodensification techniques. Through the maturity of Vibro-compaction processes, it was discovered it was uneconomical to develop machinery capable of breaking the bonds of cohesive materials. Thus, rather than fight the soil properties of silts, clays, and fines, a technique was developed to bypass the in-situ properties, and stone columns were discovered.

Stone columns are commonly used in soft, normally consolidated compressible clays, thin peat layers, saturated silts, and all laminated alluvial or esturaine soils. Stone columns have been formed successfully in soils with undrained cohesive strengths as low as 7 kiloNewtons per meter squared (Bell, 1975). The derivation of this technique lies in the inter-particle force between cohesive soils. This force cannot be overcome by conventional vibrational forces. Therefore, it is necessary to introduce material that is compactible by vibrating methods into the in-situ material. The theory is then based upon the local substitution of soil at the compaction points (Besancon, 1982).

Both Vibro-compaction and stone column construction techniques are very similar in procedure. Both incorporate the vibroflot as the main piece of equipment. As discussed earlier, the penetration operation is identical. Once the required depth is achieved, the hole is "flushed". That is, the jets of the vibroflot are fully opened. This "strong washing" forms an open cylinder in the cohesive soil. Similarly in stone column placement, it is important the hole remains open and does not collapse. Collapsing of the sides can cause contamination of the stone column. Once the whole is clean and clear, gravel backfill is dumped into the hole in increments of 0.4 meters to 0.8 meters (Mitchell, 1981). The gravel fill used to form the stone columns varies in size from 20 millimeters to 75 millimeters. As the gravel is placed, the probe simultaneously compacts the material, which in turn, displaces the gravel radially into the soft soil. The probe can be withdrawn at a rate of 0.3 meters per minute. Depending primarily on the strength of the subsurface soils, a 0.8 meter to 1.5 meter diameter finished column is

constructed (Bachus, 1989). The amount of gravel fill consumed, and vibrator power (measured by amperage) during compaction, are recorded to access the uniformity of compaction and size of the completed stone column.

In general, stone columns are usually constructed to stabilize or improve a site rather than to provide a single structural foundation. Once an individual stone column is complete, the equipment is relocated and the process repeated at an adjacent location. Spacing is a function of the in-situ soil properties, however, it generally varies between 2 meters and 3 meters, resulting in a 20% to 35% soil replacement in treated areas (Bachus, 1989). Typical production rates vary according to depth stabilized, however, as a rule of thumb, 9 meters to 18 meters per hour is average (Welsh, 1991).

1. Process

The merit of a stone column is the ability to adapt itself to the load so that collapse is prevented (Datye, 1982). Stone columns are very effective in preventing foundation failure, however, settlements may still be large. Stone columns are best employed when the settlement of the foundation system is within the tolerance limit of the structural settlement. Stone columns can reduce settlements by over 40% when compared with settlements of untreated areas (Datye, 1982).

Stone columns perform three functions meriting their use. They stabilize the ground by way of reinforcement, drag forces on the stone column are mobilized immediately, and the drainage paths of the stone columns make the consolidation process very rapid.

Stone column systems in soft, compressible soils are somewhat like pile foundations, except pile caps, reinforcement, structural connections, and deep penetration into firm strata are not required. In addition, stone columns are compressible and will deform to mobilized strength and relieve stresses during load application. When used for support, the bearing capacity and settlement are of primary concern. When used for stability purposes in embankments or slopes, the shear strength of the columns, as shown in Figure 14, is of primary concern (Mitchell, 1981).

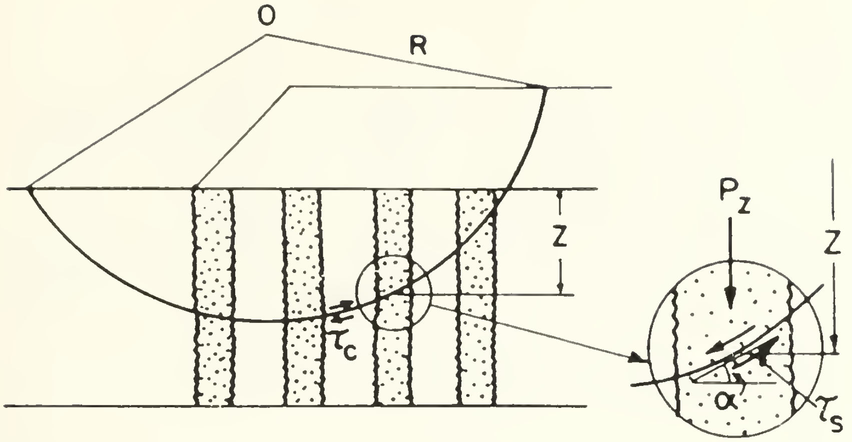


Figure 14. Shear resistance of stone columns in slope stability. (Mitchell, 1981)

However, the emphasis of this paper is the use of stone columns to reduce settlements and increase bearing capacities.

Several methods for determining the bearing capacity and load-settlement behavior of stone column foundations, ranging from experience based methods to sophisticated finite-element analysis, have been proposed.

Stone column design is based on theoretical analysis, scale model testing, and field performance. It is important to note the majority of design emphasis has come from the later methods (i.e., scale model testing and field performance). To design the stone columns, it is first important to identify the significant modes of failure. Three failure modes have been identified (Datye, 1982).

- Bulging of the stone column involving plastic failure of the expanding cylindrical column.
- Shallow shear.
- Shear failure in end bearing or in skin friction.

In design, failure in the second mode (i.e., shallow shear) is easily overcome. Remembering that soil stiffness and strength increase with depth, an adequate layer of granular material may be placed over the treated area, thus preventing shallow failure. For design purposes, layer thicknesses between 1 meter and 2 meters should be adequate (Madhav, 1978). Again using conventional pile theory, failure in end bearing or skin friction can be avoided. Therefore, for design purposes, the failure mode most difficult to predict and overcome is "bulging". With this in mind, the following discussion is presented.

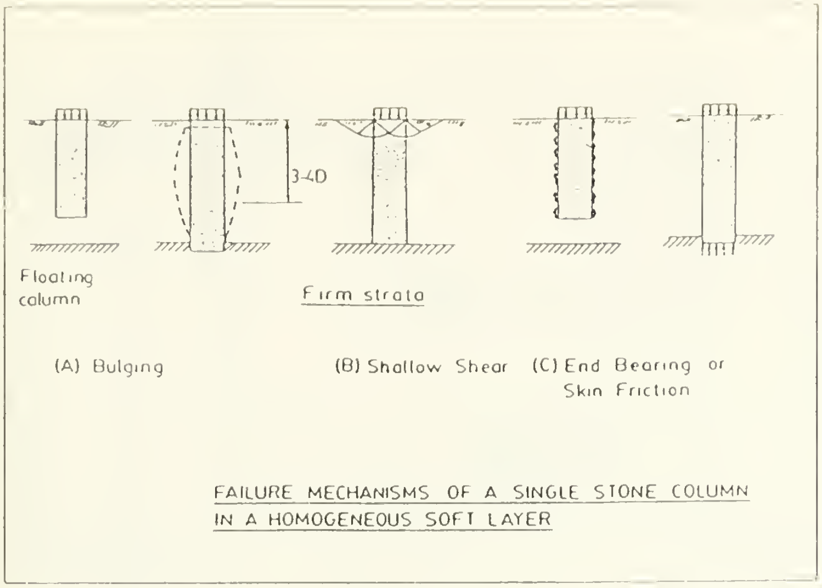

3-4D
Floating column
Firm strata
(A) Bulging
(B) Shallow Shear
(C) End Bearing or Skin Friction
FAILURE MECHANISMS OF A SINGLE STONE COLUMN IN A HOMOGENEOUS SOFT LAYER

Figure 15. Various failure mechanisms of stone columns. (Datye, 1982)

2. Design

As discussed earlier, stone columns are used when the in-situ soil is not suitable for densification through Vibro-compaction techniques. As evident from previous discussions, these soils generally contain greater than 25% fines passing the no. 200 sieve. The stone column is therefore similar to a conventional pile, keeping in mind that the surrounded cohesive soil is not significantly modified by the vibrations caused during placement. It is important to remember the stone column has no mechanical resistance by itself, and can only develop its strength due to lateral pressure reaction, provided by the surrounding soil (Besancon, 1982). Therefore, the stone column must develop a force against the surrounding soil in order to mobilize the passive earth pressure. In order to mobilize the passive earth pressure, the column deforms outward in all directions, or "bulges". The degree of bulging is the determining factor in whether the column is stable or fails. Hughes et al., (1975) have shown bulging is most likely to occur near the top, due to the lateral confining pressures being minimum there. The radial deformation decreases with depth, and appears negligible beyond a depth greater than twice the pile diameter. Figure 16 depicts the characteristic "bulging" stone column.

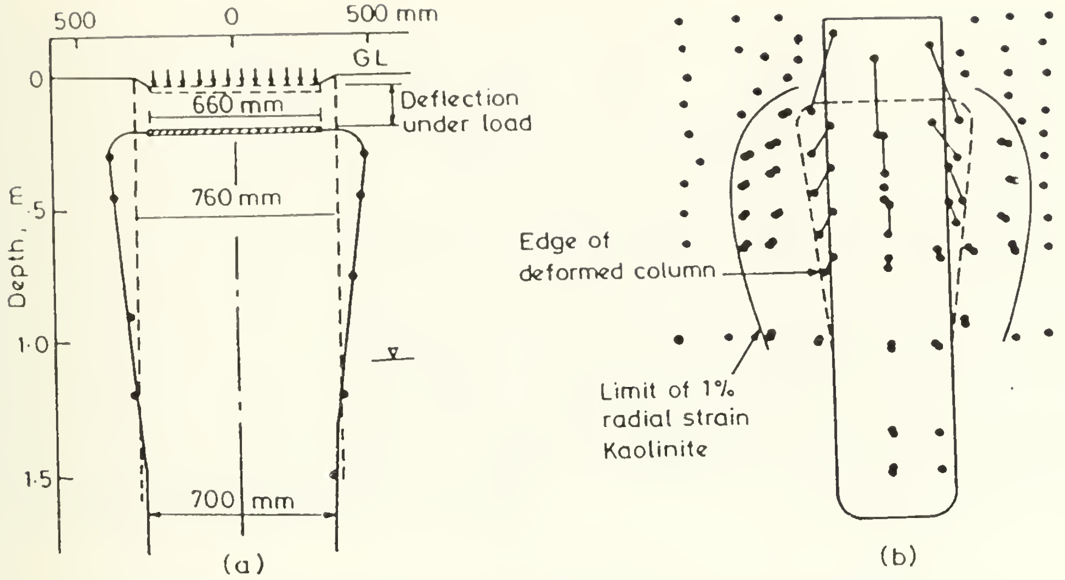

500
0
500 mm
GL
0
660 mm
Deflection under load
760 mm
Depth, m
.5
1.0
1.5
700 mm
(a)
Edge of deformed column
Limit of 1% radial strain Kaolinite
(b)

Figure 16. Typical stone column deflection. (Hughes, 1975)

3. Bearing/Load Capacity

Based on the mobilization of passive pressure, as discussed earlier, the following two formulas are presented: (Greenwood, 1970).

As mentioned, if the pile material is compressed axially, it will naturally seek to expand radially, thus causing the surrounding cohesive material to mobilize passive earth pressure. The passive resistance can be expressed as follows:

$$\sigma_R = \gamma z k_{pc} + 2C\sqrt{k_{pc}}$$

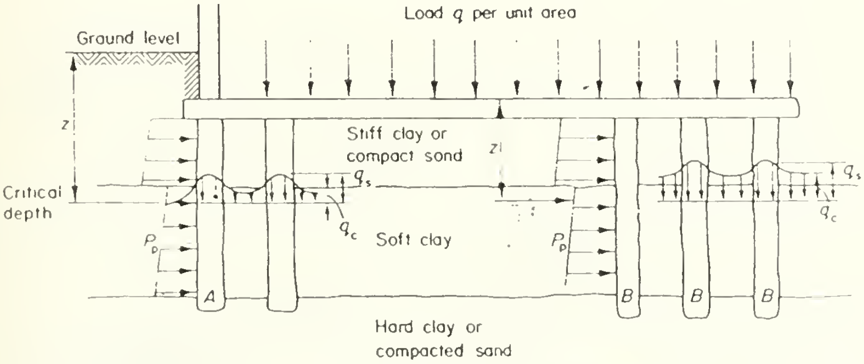
where σ_R = passive resistance of the soil
γ = unit weight of the soil
C = cohesion of the clay
k_{pc} = the Rankine passive soil coefficient
z = depth.

Using the value obtained for passive resistance above, the ultimate stress that can be carried by the stone column is:

$$q_u = \sigma_R k_{pc}$$

where q_u = ultimate stress
σ_R = passive resistance of the soil
$k_{pc} = \tan^2(45 + \phi/2)$
and ϕ is the angle of shearing resistance of the stone column material.

Since 1970, similar theories have been presented using the same basic concept, however, using different terminology. In 1975 F.G. Bell described the stone column as an axial loaded frictional material supported by the passive resistance of the surrounding cohesive material. The importance of a thorough and competent ground investigation is addressed in the approach. Unlike Hughes, Bell's method examines the entire stone column length and pinpoints the area of minimum passive resistance. This is done through a plot of the profile of passive resistance determined by the soil properties at varying depths. In addition to a complete set of boring logs, the stress-strain relationship, and maximum and minimum friction angle for the compacted column must also by known or assumed. Excess pore water pressures generated by the load are considered negligible. Keeping in mind the relative close spacing of the columns, the free draining column material, and the loading rate, this is a fair assumption. Figure 17 depicts a typical structure with corresponding passive restraints.



At critical depth, the average stress on the clay is q_c and on the column q_s. Let $q_c = xq$; passive restraint in the critical zone where columns are weakest is then given by:

$$P_p = \gamma z K_{pc} + 2c\sqrt{K_{pc}} \qquad \text{and} \qquad P_p = (\gamma z + xq)K_{pc} + 2c\sqrt{K_{pc}}$$

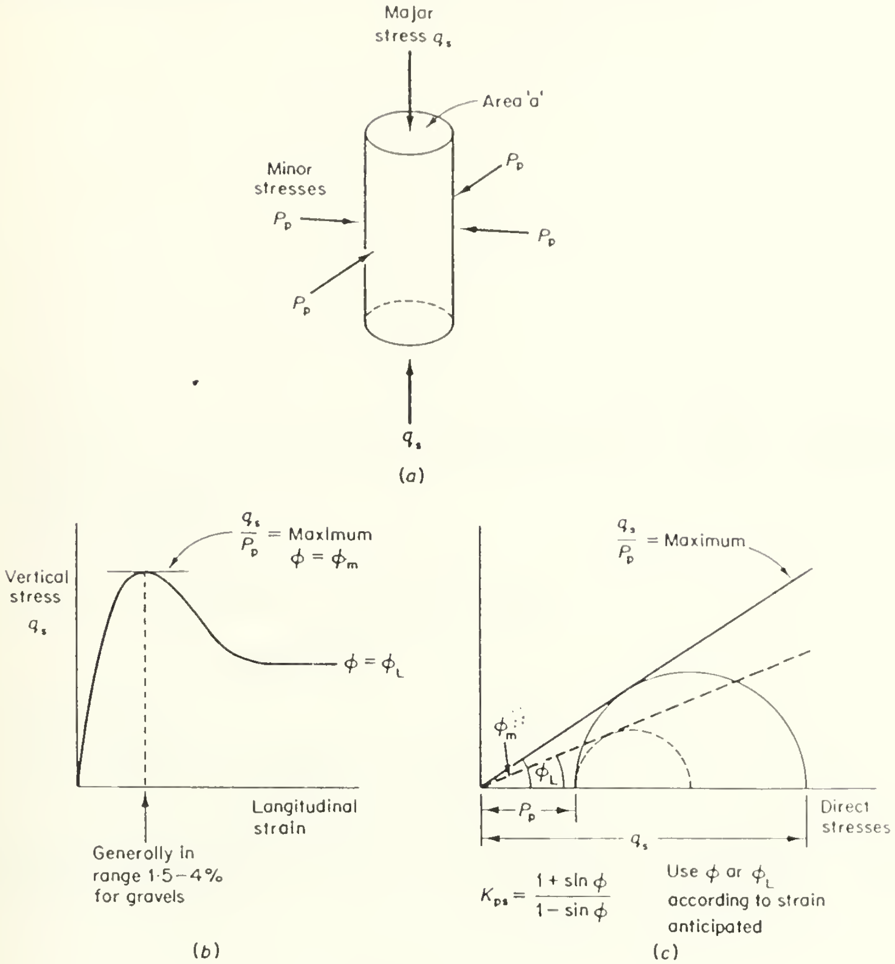
(A at *periphery* or under *narrow footings*) (B: under *central areas* of *wide foundations*)

Figure 17. Stone Column Design - Passive Restraint. (Bell, 1975)

A significant difference between Bell and Hughes' development stem from the in-situ material. Hughes assumed a homogeneous strata, whereas, Bell emphasizes the boring log and layered strata.

Maximum column bearing capacity is achieved when the ratio of applied stress on the column to passive restraint at the critical depth, is at its maximum. Passive restraint is fully developed at relatively small radial strain because of the mode of column construction in which backfill is packed into the bore. Radial shear strains in the soil associated with development of passive resistance are greatest where passive strength is least. Peak stress ratio is therefore first achieved at critical depth. Elsewhere in the column, radial strains will be smaller and the stress ratio will not have

reached its peak value. Figure 18 shows this graphically.


Major stress q_s
Area 'a'
Minor stresses
P_p
P_p
P_p
P_p
q_s
(a)
$\frac{q_s}{P_p}$ = Maximum
$\phi = \phi_m$
Vertical stress q_s
$\phi = \phi_L$
Longitudinal strain
Generally in range 1·5–4% for gravels
(b)
$\frac{q_s}{P_p}$ = Maximum
ϕ_m
ϕ_L
P_p
q_s
Direct stresses
$K_{ps} = \frac{1+\sin\phi}{1-\sin\phi}$
Use ϕ or ϕ_L according to strain anticipated
(c)

$$q_s a = K_{ps} P_p a = K_{ps} a(\gamma z K_{pc} + 2c\sqrt{K_{pc}} + xqK'_{pc})$$

Figure 18. Estimation of column bearing capacity.
a.) Critical zone of column stressed triaxially
b.) Stress-Strain diagram
c.) Mohr diagram
(Bell, 1975)

The maximum load that can be supported by the column cannot exceed the peak stress q_s multiplied by the estimated column plan area, which should include a suitable margin for variation occurring in practice. A check must be made to ensure that soil below critical depth can support the load as a pile. (Bell, 1975)

Besancon (1987) equates the stone column behavior to that of a granular sample during a triaxial test. Based on the triaxial test, Besancon developed

stone column bearing capacity as follows, based on Figure 19.

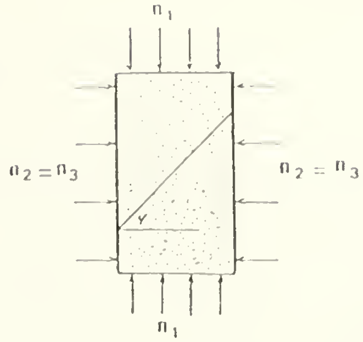


Figure 19. Stresses on granular sample during Triaxial Test. (Besancon, 1982)

where n_1 = total pressure
n_2 = lateral pressure

At failure, the total pressure (n_1) will equal:

$$n_1 = n_3k_p+2Ck_p$$

where $K_p = \text{Tan}^2(45+{}^{\Phi}/_{2})$
and Φ is the angle of internal friction.

If a stone column is installed correctly (i.e., no contamination by cohesive material) the stone column is a cohesionless material, therefore, the previous equation reduces to:

$$n_1 = n_3k_p$$

$$\text{or } n_1 = P_1k_p$$

where P_1 = the limiting lateral pressure of the soil.

To get an indication of the value of lateral pressure (n_3) provided by the in-situ material, simple vane tests, penetrometer tests, or pressiometer tests can be performed prior to design. It is also possible to assume a value based on information obtained in the soil report.

As can be seen from the previous three theories (although all similar), bearing capacity of the stone column is a function of the angle of internal friction of the column material and the passive pressure applied by the in-situ material. The angle of internal friction in a stone column generally ranges from 40 degrees to 45 degrees depending on the material used (Bell, 1975). However, to include a factor of safety it is general practice to use 38 degrees for design purposes (Besancon, 1982). This is the lowest value to

date ever recorded in a stone column. Based on the above information, a simplified design formula is as follows:

$$n_l = 4P_l \quad \text{(Besancon, 1982)}$$

where n_l = total vertical pressure
P_l = limiting lateral pressure.

In 1984, D.A. Greenwood presented an equation to determine the ultimate bearing capacity of a single stone column. It is understood, in clay soils or essentially clayey fills, the limit of acceptable settlement will be reached well before the ultimate bearing capacity of the stone column. Therefore, design will usually be based on settlements. As a general guide the shear strength (c_u) of cohesive material should be at least 20 kiloNewton per meter squared for stone columns to be effective, although in special circumstances, soils with shear strength as little as 15 kiloNewton per meter squared have been treated.

The ultimate bearing capacity of a single stone column can be obtained from:

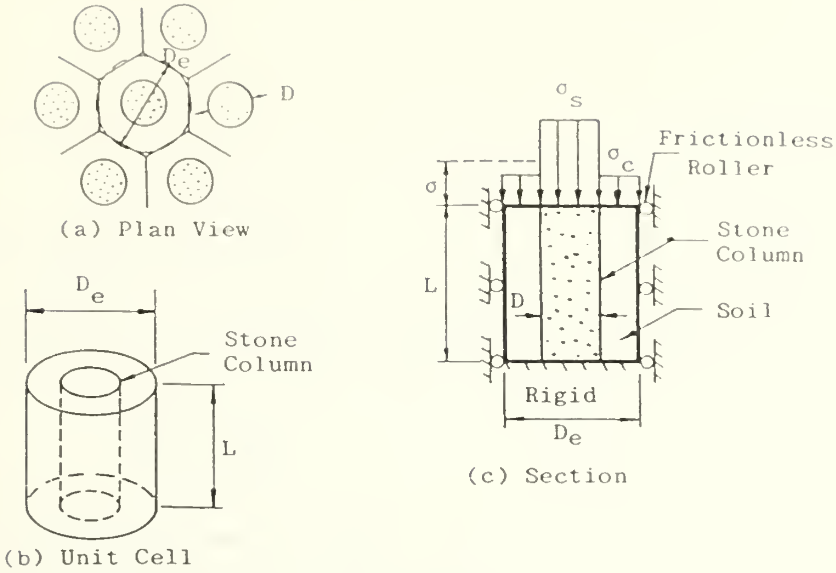
$$\sigma_{vc} = \tan^2(45 + \Phi/2)(Fc_u + \sigma'_{r0s} - U_o)$$

where σ'_{r0s} = lateral pressure including surcharge
F = multiplier (4 as suggested by Gibson & Anderson)
U_o = 0 when column is effective in reducing pore water pressure
C_u = undrained shear strength for small groups
= C' for large column groups.

With bearing capacity discussed, it is necessary to investigate spacing as it relates to settlements. Although neither of the three (i.e., bearing capacity, spacing, and settlement) should be excluded from the overall design, it is important to segregate them for discussion purposes.

4. Spacing and Settlement

When considering the spacing and settlements of stone columns placed in a soft subgrade, it is important to develop a model on which to base all types of performance: Due to the complexity of the design, it is economically unfeasible to model all possible spacing and load combinations. Therefore, many engineers have adopted the use of a "unit cell" to model the effects of stone columns placed in soft strata. This concept of a "unit cell" is shown in Figure 20.



(a) Plan View

(b) Unit Cell

(c) Section

Figure 20. Unit cell idealization. (Bachus, 1989)

An important factor of stone column design is the amount of soil replaced by the stone. This parameter is considered in design, and also measured in the field during actual placement of the stone column. The area replacement ratio (Bachus, 1989) is defined as follows:

$$a_s = A_s/A$$

where a_s = the area replacement ratio
A_s = the area of stone column
A = the total area in the unit cell.

The area replacement ratio can also be defined as

$$a_s = 0.907\ (D/S)^2$$

where D = the column diameter
S = the column spacing.

The constant term (i.e., 0.907) is based on the pattern used. In this case the typical equilateral triangle is used.

The 'unit cell' concept is useful in analysis of the performance of the stone columns, it will be implemented throughout the following discussions.

In addition to the 'unit cell' approach, the concept of stress concentration is important to understand. Placing a uniform load (i.e., an embankment or foundation load) over stone columns will cause a concentration of stress within the stone column. This is due largely to the varying stiffness between the stone and subgrade soils. Bachus defines the stress

concentration factor as:

$$n = \sigma_s / \sigma_c$$

where σ_s = the stress in the stone
σ_c = the stress in the subgrade soil due to the additional load.

Since vertical equilibrium must be maintained, the actual incremental stress increase in the stone and subgrade is as follows:

$$\sigma_s = u_s \sigma$$
$$\sigma_c = u_c \sigma$$

where σ = the average applied vertical stress

u_s and u_c are the ratios of increased stress in the stone and clay respectively.

Stress concentration in the stone upon initial loading of the stone column and surrounding soil, initiates a rather complex interaction of stone and soil. The response of the stone to the high stresses is to bulge laterally and thus also undergo vertical movement. This motion is restrained by the lateral resistance and confinement of the surrounding soil. The net response is an enlargement of the stone column in the upper reaches of the foundation and a complementary vertical deflection of the composite column/foundation. From this conceptual point of view, it is apparent that for stone columns to develop load resistance, the composite stone/subgrade must deform vertically. Therefore, while the stone columns may reduce settlements, their use will not eliminate deformation. This important factor must not be overlooked. Design methods for predicting the settlement of stone columns vary from empirical techniques to a rather complex yet complete incremental analysis. All methods consider the stress concentration concept, the area replacement ratio and the stiffness of the subgrade soils. Direct comparison of each method, however, shows reasonably consistent trends.

Stone columns in soft compressible soils are somewhat like pile foundations, except that pile caps, structural connections, and deep penetration into underlying firm strata are not required, and the stone columns, are of course, more compressible. When used in lieu of pile foundations settlements are of primary concern (Mitchell, 1981).

For structures with small factors of safety on settlements (i.e. close restrictions on non-uniform settlements) it is assumed the stone columns carry the entire load. This is a very conservative approach since it is known (to be discussed later) that as a stone column approaches maximum load capacity, an increased share of the total load is carried by the surrounding soil.

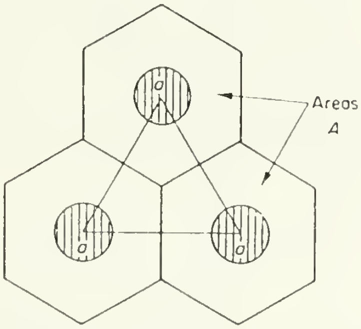
Bell, in 1975, contributed the following, relating stress-strain

characteristics with settlements. Stone column spacing is chosen so that the maximum stress ratio is not exceeded. In this way, strain is restricted to the zone of the column near the critical depth (again, critical depth being associated with the soil strata providing the least passive resistance) which, therefore, contributes the major part of the settlement. An estimate of the magnitude of strain can be made from the stress-strain diagram and estimating that the length of the critical depth is approximately one to two column diameters.

For compact gravels (i.e., stone columns) the vertical strain corresponding to the maximum stress ratio is usually in the 1.5 - 4.0% range. For lesser strains, a large stress change produces only a small change in strain, and contributions to settlement outside the critical zone of the column are insignificant. On this basis, it is reasonable to assume working-load settlements are restricted to 20 - 40 millimeters (Bell, 1975).

As was discussed earlier, the loaded stone column will dilate (i.e., bulge). When this occurs, vertical strains will be less than twice the radial strains. This outward movement of the column is enough to mobilize the passive resistance of the surrounding soil.

Using Figure 21, along with the associated equations, the stresses in a column may be estimated. Thus solving for "x", the stress and bearing capacity can be determined. Since columns and surrounding soil will settle together, the magnitude of settlement may be estimated conventionally from the average stresses "q_c" on the soil between columns.



Total area of foundation = ΣA
Total load stresses on area $A = q_c$
Average stress on soil area = $(A - a)$
Average stress on column area $a = q_s$

Let $q_c = xq$
then $q_c = \frac{qA - q_s a}{(A - a)}$
and hence from Fig. 11.4:

$$q_c = xq = \frac{qA - K_{ps}\, a(\gamma z K_{pc} + 2c\sqrt{K_{pc}} + xqK_{pc})}{(A - a)}$$

from which x is obtained. The stresses and bearing capacity can thus be determined. Since columns and soil settle together the magnitude of settlement may be estimated conventionally from the average stresses q_c on the soil between columns

Figure 21. Bearing capacity equation. (Bell, 1975)

Bell's semi-empirical design method ensures column spacing is determined rationally from the soil properties that are easily obtained from the soil report. It does not give stress-strain relationships, but only suggests limits for safe design. This method gives an indication of the likely limiting settlement based on working-loads.

In 1989, Bachus incorporated the stress factor and unit cell into one dimension settlement theory, and presented the Equilibrium Method. As part of the Equilibrium Method, the settlement ratio was defined.

$$S_T/S = 1/[1+(n+1)a_s] = u_c$$

where S_T = the settlement of treated ground
S = the total settlement of the untreated material.

The settlement ratio considers the reduction in volume of the compressible material due to the replacement by stone, and the reduction of stress on the compressible material due to the stone. This equation generally describes the curves shown in Figure 22 and can be considered the upper bound of anticipated soil improvement.

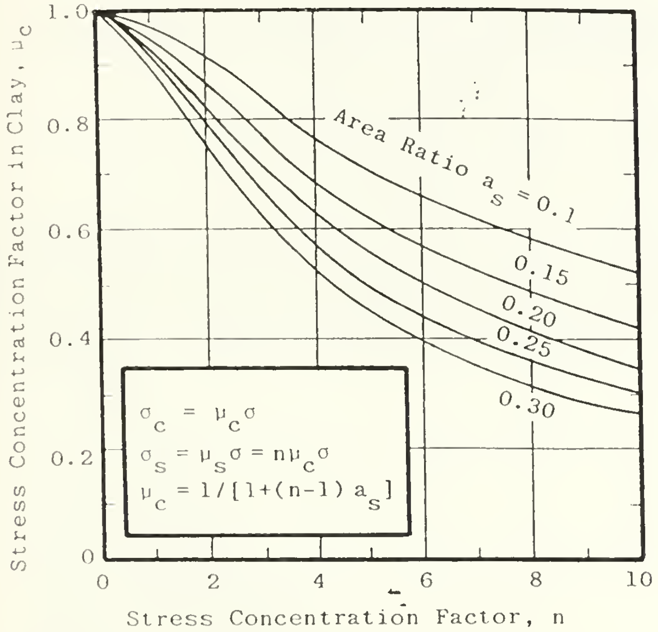


Figure 22. Variation of stress concentration factor, n. (Bachus, 1989)

At large replacement ratios, the stress in the clay is minimized, but the costs associated with the stone column foundation and disposal of replaced material become prohibitive, although the settlement is practically reduced. Conversely, the lower the area replacement ratio, the smaller the effect on

settlement (Bachus, 1989).

A comparison of this relatively simple approach to that of an actual field performance is shown in Figure 23. The Equilibrium Method and Greenwood's recommendations, which are based on field experience, are generally bounded by the results for n = 5-10. Therefore, the Equilibrium Method can be confidently used as an upper bound for settlement predictions.

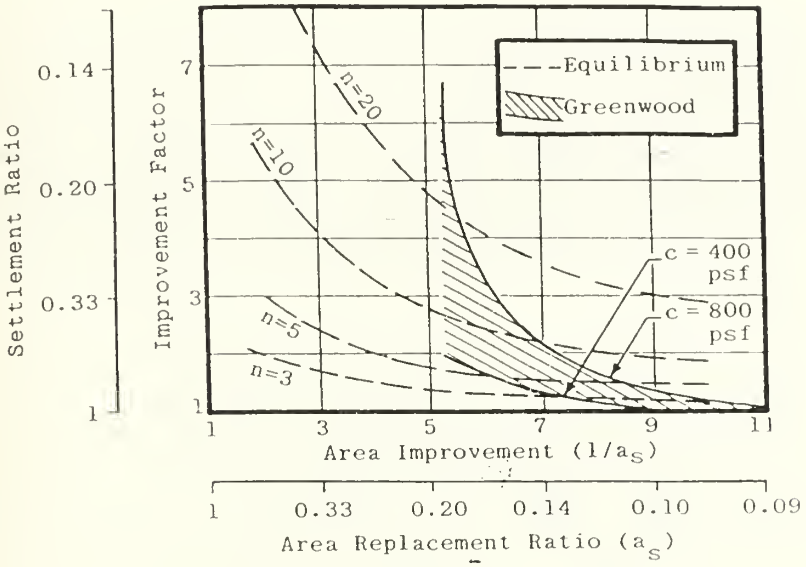


Figure 23. Comparison of methods for settlement reduction. (Bachus, 1989)

Having mentioned the above, probably the most practical method for computing settlements is Priebe's method (Greenwood, 1984). Figure 24 illustrates this point.

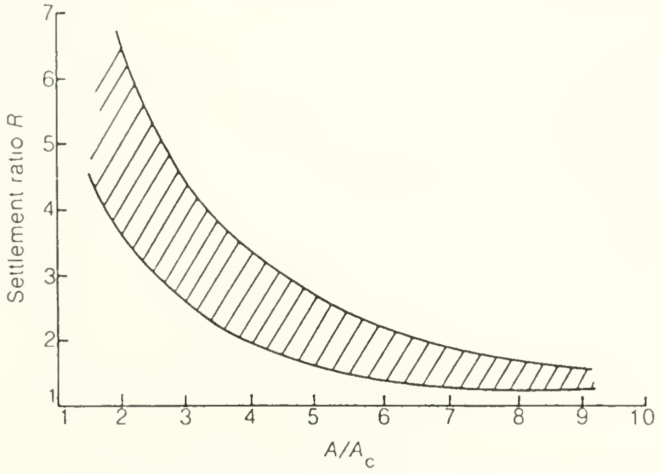


Figure 24. Priebe's Method. (Greenwood, 1984)

From this figure an estimate of the settlement of the loaded area can be obtained. The cross-sectional area of the stone column is of critical importance in relation to load carrying capacity. This requires close control of the sitework, with detailed documentation. For an initial approximation, it can usually be assumed that for isolated shallow footings, settlements will be reduced by 50% compared with untreated ground.

5. Contact Pressure Distribution

As mentioned earlier, most engineers take an extremely cautious approach when designing stone columns. By assuming the stone column carries the entire load, engineers have built in a safety factor that increases as the load applied increases. Figure 25 depicts the results from a typical test designed to measure contact pressure under a rigid footing. The test was performed over an industrial waste dump. The soil consisted of finely ground, spherical silica particles, arising from a glass manufacturer. The spherical silica fell entirely within the fine silt gradation range.

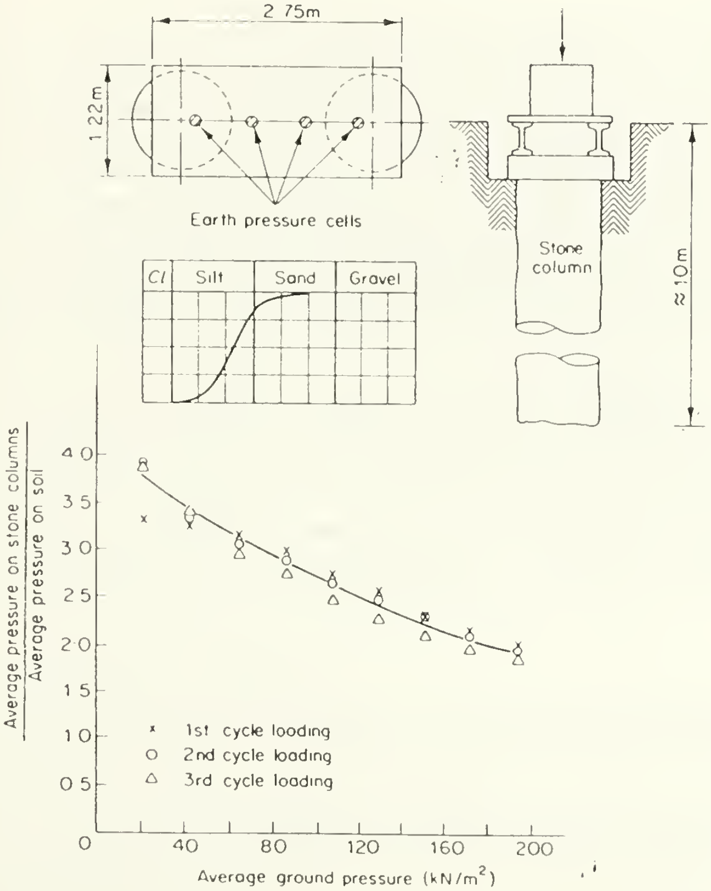


Figure 25. Loading test for contact pressure. (Bell, 1975)

Earth pressure cells were built into a precast concrete foundation. The cells were spaced evenly over the columns and the soil.

Examining the results (i.e., average ground pressure vs. the ratio of earth pressures) highlights some significant findings.

First, it is evident that as the load on the foundation increases, the ratio of average stone column pressure to average soil pressure decreases. That is to say, as the load increases on the foundation, the load carried by the stone column decreases while the load carried by the soil increases. This is significant, in that the columns carry virtually the entire initial load when the footing load is small, and the soil takes on an increasing share as the load increases. Thus at high stress levels, the contact pressure is comparatively uniform (Bell, 1975). From an economic point of view, this is significant with respect to reinforcement in both the footings and columns. Because of the relatively uniform contact pressure, nominal bending moments are experienced in the footing, thus decreasing the need for excessive reinforcement.

Secondly, in this test, the critical zone was close to the base of the footing, enabling a close correlation between the critical zone and earth contact pressure to be made. Using the critical zone theory presented earlier (Bell, 1975), the calculated ratio of vertical stresses on the column and soil at maximum bearing load of 193 kiloNewtons per meter squared was 2.2. The actual test results indicated a ratio of 2.0, thus enabling the generalized correlation to be made. Maximum settlement experienced by the stone column was 15 millimeters. This amount of settlement was enough to induce significant radial strains large enough to fully mobilize the passive restraint of the soil.

Compressibility of the stone column (during loading) is a distinct advantage over a conventional rigid pile system. The uniform contact pressure experienced by the stone column/rigid footing system is advantageous for uniform settlements. Furthermore, if one of the stone columns would fail, the lateral distribution of contact pressure would continue to be wide, thus causing only a slight dip in the distribution. Most buildings are designed to easily accommodate such limited settlements.

F. Stone Columns and Slope Stability

In addition to reducing settlements and increasing bearing capacity for foundation use, stone columns can also be used for slope stabilization.

The majority of interest centered in slope stability today concerns the use of geotextiles and geosynthetics, however, stone columns present a viable and economical alternative.

As slope stability theory dictates, in cohesive soils deep seated

failures are most likely to occur. Therefore, stone columns are a practical candidate to be used in the stability of cohesive slopes. When used for stability purposes under embankments or slopes, as shown in Figure 26, the shear strength of the columns is of primary interest (Mitchell, 1981).

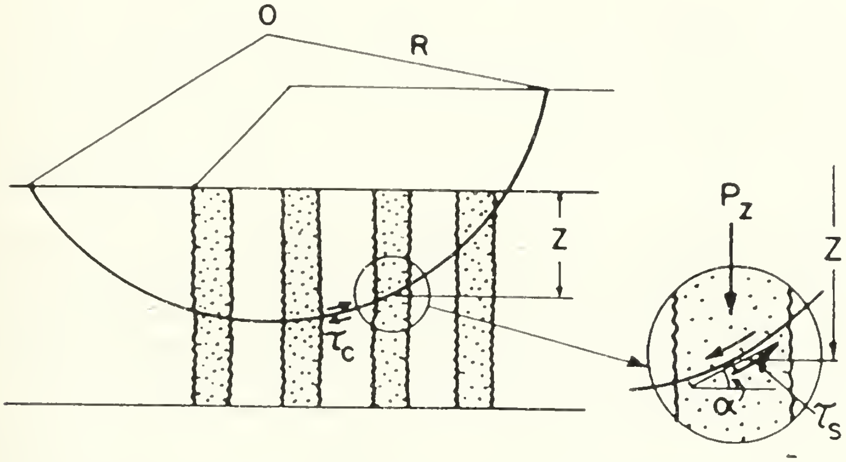


Figure 26. Stone columns used for slope stability. (Mitchell, 1981)

The shear resistance along the failure surface is a function of the internal friction angle, as well as the normal forces and friction coefficient. The material used in the columns can be considered to have zero effective cohesion (c') and friction angles between 35 degrees and 40 degrees, the same properties considered when used for foundation reinforcement. The increased friction angle in the stone column increases the overall friction along the failure surface or slip plane, and thus increases the factor of safety against failure. Although simple in theory, most engineers have been caught in the wave of the geotextile movement and have forgotten the role stone columns can play in slope stability. Analysis of the reinforcing effect of stone columns in stability applications for slopes and embankments is usually done on the basis of composite shear strength (Mitchell, 1981). The composite shear strength is based on the undrained shear strength of the cohesive material, the transverse shear strength of the columns, and the replacement ratio. The transverse shear strength depends on the normal stress at the failure surface located along a line of interaction between the soft cohesive material and the column. This value is not easily obtained and assumptions and approximations are required. Aboshi, in 1979 presented the following for the composite shearing resistance (τ) located at any point along the circular sliding surfaces:

$$\tau = (1-a_s)\tau_c + a_s\tau_s\cos\alpha$$

where $\tau_s = (P_z\tan\phi_s)(\cos\alpha)$.
$P_z = \gamma'_s Z + \sigma_z u_s$

Once the composite shear strength is determined, the factor of safety for slope stability can be determined.

G. Practical Considerations

Like it or not, most geotechnical engineers will accept the fact, soil borings will not give an accurate description of underlying surface conditions for an entire construction site. Because soil borings are only representative at the exact location taken and most projects do not budget for in depth site surveys, unforeseen conditions are commonly encountered. Unlike in theory, soils rarely have uniform characteristics throughout a site, except when classified in the broadest of terms. Not only do lenses of various strata occur throughout the sites, broad classifications of soils often encompass too large a spectrum of materials. It is for this reason, that knowledge of the formation and history of superficial deposits can aid in the subsurface portions on any project.

In consideration of the above circumstances, to be successful, any geotechnical process must be able to cope with variations without alteration of projected structural design (Bell, 1975). For this reason, when considering Vibro-compaction vs. Vibro-replacement, Vibro-replacement or stone columns invariably are chosen. The unexpected presence of a lens of soft clay in an alluvial sand deposit can be embarrassing, especially if the lens is substantial (i.e., has overall thickness greater than the diameter of the intended compaction zone). However, as shown previously, the effects on overall settlement can be negligible when stone columns are used. The column will form a very stiff structure through the compressible material; lenses of substantial thickness should be detected in site investigation. Figure 27 depicts both situations.

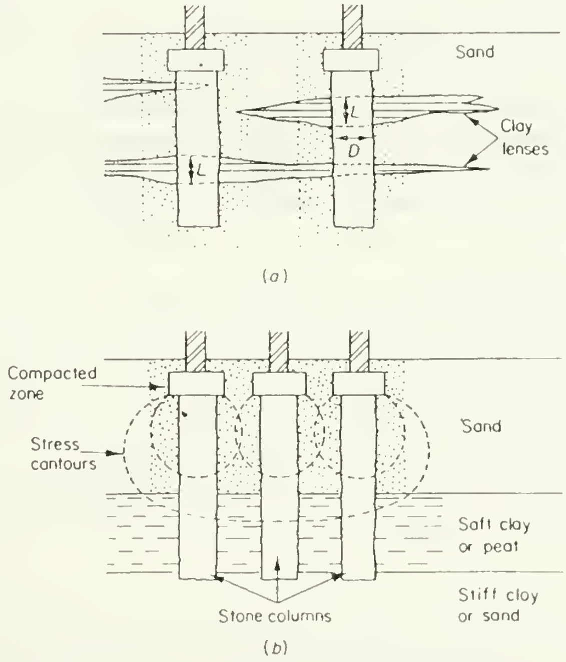


Figure 27. Treatment of Mixed Clays and Sands (Bell, 1975)

Excluding ideal situations (i.e., in-situ ideal sands with perfect gradation) stone columns require less operator skill than Vibro-compaction techniques. Stone columns can be constructed quicker (10-30 minutes per column by an experienced crew) with less oversight than Vibro-compaction techniques. In addition, with the discovery of a significantly large zone of poor soil, additional stone columns can be constructed without a serious delay in the project schedule. However, the trend in todays designs is to include multiple ground improvement techniques on one site in lieu of designing a single technique for the entire site (Welsh, 1991). This will be shown in the case study of the Trident Submarine Facility in Kings Bay, Georgia.

H. Environmental Considerations

Although the use of ground improvement techniques is a way to recover once unsuitable land, it is not without its own environmental concerns. Vibro-replacement requires about 35 cubic meters per hour of a water supply (difficult to obtain in developing countries or rural areas) and produces an effluent of water and suspended solids. The solids consist primarily of silts and clays. Most currently available "packages" include sedimentation tanks and flocculating chemicals to clarify the effluent to the order of 25-40 parts per million of solids (Welsh, 1991). It is important to check local drainage authorities and environmental laws concerning the required discharge permits.

I. Additional Considerations

It is important to keep in mind stone columns require a well graded coarse granular backfill (i.e., usually between 0.5 and 3 inches in diameter) available on site (NAVFAC, DM 7.3). Each vibroflot can consume 300-500 tons per day. It is important for economy and quality that the supply be kept constant in order to keep the vibroflot continuously working. Delays in stone column construction can adversely affect the interparticle attractions in the surrounding soil fabric, thus requiring additional jetting and stone to restore stability (Bell, 1975).

While theory dictates a well graded material will have greater mechanical strength than a uniform stone, the practical difference is insignificant during wet operations, or through pore water dissipation, the coarser fines from the bore migrate to the voids in the larger imported material. Upon close examination, the filling normally consists of coarse silt and fine sand which becomes coarser and cleaner towards the central core of the column (Besancon, 1982). In addition to the size of the fill material, the chemical make-up must resist disintegration from any cause during its intended useful life.

Efficient operations placing stone columns require a sound working platform. Since stone columns are usually prescribed for cohesive materials adequate bearing must be provided for the crane. The crane and related equipment may have a pull up to five times the weight of the machine when

withdrawing the vibroflot (Bell, 1975). When pricing for projects it is important to provide for and price in the design for surfacing in working areas to support the tracked equipment.

J. Conclusion

Ground modification includes processes for strengthening weak superficial soils to allow the use of conventional shallow building foundations. The techniques described have undergone constant refinement and are now considered viable alternatives to deep foundations in unstable soil

The current trend in geotechnical engineering is toward geotextiles and geosynthetics. It is important to keep in mind the value of both Vibro-compaction and Vibro-replacement in todays industry, both still have a prominent place among all ground improvement techniques and never should be overshadowed. The steady evolution of Vibro-compaction from vibratory densification of loose sands to strengthening weak clays by reinforcing them with columns of gravel has ensured its practical reliability. In addition, the speed and simplicity of treatment allows any unforeseen conditions to be dealt with quickly without serious delays.

Vibrodensification techniques will continue to be a viable alternative to deep foundations. With the amount of available land shrinking, engineers must look to alternatives, vibrodensification can fill the void left by unsuitable project sites.

II. Case Study

Soil Improvement at the Trident Submarine Facility, Kings Bay, Georgia.

Soil Improvement at the Trident Submarine Facility, Kings Bay, Georgia

A. Introduction

In November of 1976, the Secretary of the Navy announced plans for the construction of the $1.7 billion Trident Atlantic Coast Strategic Submarine Base. The location selected was Kings Bay, Georgia. The new base would support the fleet of Trident class submarines responsible for patrolling the Atlantic strategic area. The base would include facilities for mooring submarines, crew training, weapons handling, and storage, maintenance and repairs, missile assembly and inspection, storage magazines, housing, and related administrative and support buildings.

The site selection concluded several years of detailed studies of various sites along the east coast. Considered throughout the analysis were operational capabilities, costs, environmental impacts, social and economical impacts on the local community, and political considerations. Unfortunately in todays government, political aspirations can weigh heavy in the decision making process. Although not ideal from a subsurface and geotechnical point of view, Kings Bay, Georgia was chosen and the task at hand identified.

To accomplish this monumental task, the Navy designated the Naval Facilities Engineering Command (NAVFAC) with sole responsibility. Because the project was so large, and would encompass several years, NAVFAC commissioned the office of the Officer in Charge of Construction (OICC), Trident, and located this on site to administer all aspects of construction. OICC Trident would be responsible for all aspects of construction, including planning, budgeting, contracting, designing, and administering the $1.7 billion project. The master plan divided Kings Bay into four major functional areas:

1. Waterfront
2. Industrial and Strategic Weapons
3. Personnel, Administration, and Training
4. Family Housing.

B. Geologic Setting

The Kings Bay site is located in the Lower Atlantic Coastal Plain. The soils to a depth of about 50 feet (15 m) are recent sedimentary deposits comprised of normally consolidated sands, silts, and clays. The sands were deposited in high energy environments (moving water) and the silts and clays in the lower energy environments of backwaters and lagoons. Variations in soil types are encountered over short distances. A phenomenon common to the coastal areas and present at this site is a near surface layer of dense cemented organic stained fine sand, commonly referred to as "hardpan".

Underlying the recent deposits and extending to a depth of approximately 500 feet (150 m) are the Charlton and Hawthorn formations, which are of Pliocene and Miocene age. They are over-consolidated deposits generally consisting of weak limestone and firm silty sands (marl). These are underlain by an older limestone formation.

C. Seismic History

A seismic risk analysis, performed by a geotechnical consultant, indicated that four significant earthquakes have affected the site area since 1800. The largest of these events occurred in 1886 as part of the Great Charleston South Carolina Event. Ground motion felt in the vicinity of the Kings Bay site was of intensity VI Modified Mercalli (MM). During a given 250 year period of time, the analysis indicated that the Kings Bay area could be subjected to a peak ground acceleration of 0.1g.

Since any future interruption in operations in the strategic weapons area would be unacceptable (i.e., once the base was completed and operating), the OICC Trident decided to reduce the risk of future settlement and liquefication potential to the underlying soil. Therefore, deep soil stabilization was chosen as an alternative approach to exclusive deep foundation design. The deep soil improvement for the majority of the strategic weapons area was bid in two projects.

D. Soil Tests

After the contracts were awarded, the contractor in conjunction with the Navy performed soil tests to determine the subsurface profile. Due to the sensitivity of the contract (i.e., the Navy's concern with future settlements), the Navy employed a quality control contractor to randomly check the contractor involved with the site improvements.

To establish a subsurface profile, standard penetration tests (SPT), electronic cone penetrometer tests (CPT), and dilatometer tests (DMT) were performed. A total of 92 SPT and 8 CPT were performed in the cumulative site area of 805,700 square feet. The average test was one test for every 8,000 square feet. Some tests extended as deep as 100 feet.

As mentioned previously in the site history, the initial 50 feet consisted of loose sedimentary deposits overlying generally overconsolidated limestones and stiff sands. It was the initial 50 feet that was of primary concern for this project. Table 1 presents a generalized subsurface profile of the loose deposits. It is important to keep in mind some of the project area had previously been stripped and grubbed and contained approximately 4 feet of compacted fill. These areas tended to increase the overall values in the initial 4 foot zone.

TABLE I
GENERALIZED SUBSURFACE PROFILE (PRE-TREATMENT)

Depth (Ft)	Soil Description	Typical Range of Test Values "N" Value (bpf)	 Tip Resistance (tsf)	 Modulus Value (tsf)
0 - 8	Fine Sand (SP, SP-SM)	2 - 50+	20 - 300	*
8 - 15	Cemented Organic Stained Fine Sands (SP, SP-SM)	30 - 100+	100 - 500+	*
15 - 18	Silty Fine Sand (SM)	2 - 9	5 - 50	25 - 100
18 - 50	Fine Sand with Silty Sand and Clayey Sand Layers (SP-SM, SM and SC)	1 - 40	5 - 250	100 - 1000

Figure 28 depicts the range of grain size distributions for the natural site soils. Some variation to this distribution will occur in localized lenses of silts or clays encountered in the deposits below 30 feet.

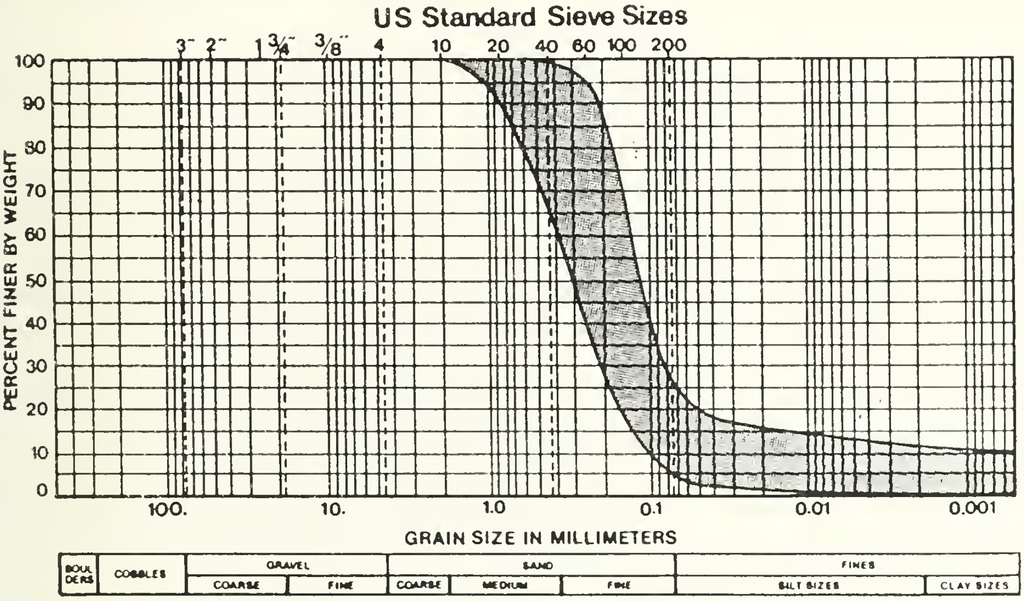


Figure 28. Range of grain size distribution of natural site soils. (Hussin, 1987)

Figure 29 superimposes the range of soils most densifiable by vibro systems over the naturally occurring soil grain size distribution.

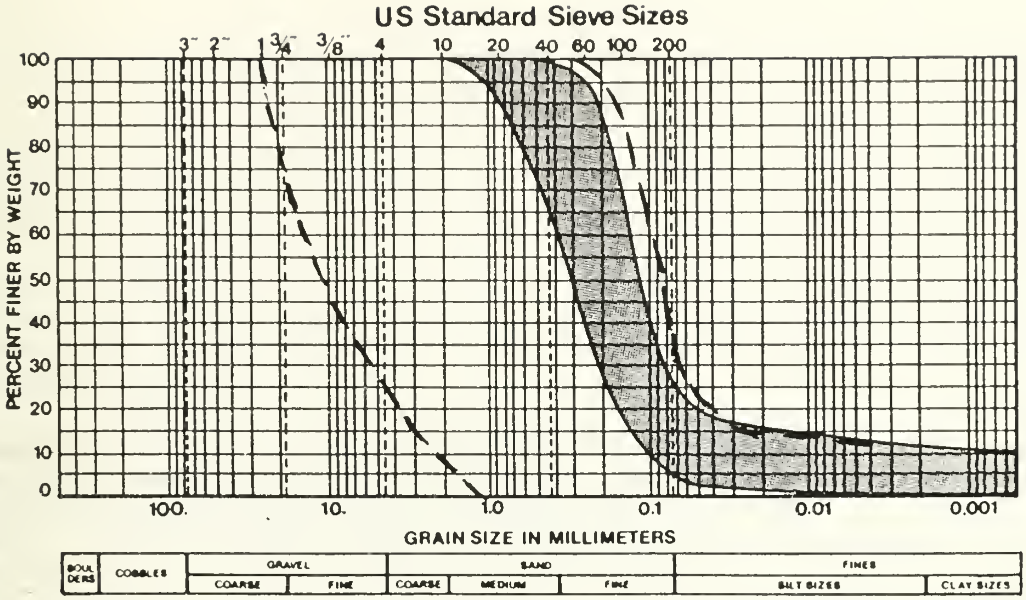


Figure 29. Range of soils densifiable by vibro-systems superimposed over the naturally occurring soil grain size distribution.

As can be seen, the soils on the site were a perfect match to those suitable for densification by composite vibro systems.

Prior to a discussion on the selection of the soil improvement techniques, it is first important to understand how the United States Navy performs projects of this magnitude.

E. Requirements and Specifications

An organization within the Department of the Navy is the Civil Engineer Corps (CEC). The CEC is responsible to the Chief of Naval Operations, as well as the Legislative Branch of Government for all Public Works, Construction, and Operation and Maintenance of U.S. Naval Facilities throughout the world. Because of the magnitude of the yearly construction budget and due to the relatively small number of CEC officers and staff, the majority of work is contracted to various design firms, construction firms, and specialty firms. The procedure for engineering design and review is similar for most projects. An architect-engineering firm is selected and it is their responsibility to subcontract and select various specialty firms, including geotechnical. In addition, the Navy often contracts quality control firms to closely monitor the contractor and supplement Government quality control programs.

Since in this case the geotechnical exploration findings were essential in deciding the basic design, the decision of the OICC was to receive the final geotechnical report with the submittal of the 35% complete design

procedure.

The Navy decided that the foundation design of the Strategic Weapons area must be as safe as economically possible. The review of the subsurface exploration reports for the Kings Bay site noted that the subsurface soils were predominately loose sands. These soils have the potential for unacceptable settlement and liquefaction as a result of possible seismic activity or vibrations due to the sudden blast of warheads or missile motors.

The OICC decided that the loose soil layers should be densified at a reasonable cost and within the time frame of the construction. Deep soil improvement was selected as the economical method to permit the safe design of shallow foundations.

The criteria was to achieve at least 65% to 70% relative density in the case of cohesionless soils. In the case of cohesive soil, the criteria was improvement of the soils profile to allow a maximum of 0.5 inch total settlement. Post-treatment SPTs and CPTs would verify the density improvement of the loose soils. DMTs would determine the soil constrained modulus value. The specifications required averaging test values over 5 foot (1.5 m) depth intervals. In soils with greater than 12% fines, the test value of any replacement material used (stone or grout) was averaged with values in the natural soil, as per the specifications.

The specifications directed the contractor to demonstrate the performance of his selected densification method in test areas. After OICC review and approval, the production work could begin.

The construction of the base had to be completed by October 1989, when the first Trident submarine was scheduled to arrive. Therefore, each aspect of the construction was on a very tight time schedule. The deep soil improvement for the SWFLANT sites was to be completed within 95 days with $3,700 per day liquidated damages. The Missile Motor Magazines Phase I was to be completed in 90 days with $1,500 per day liquidated damages. Phase II of the Missile Magazines was less critical.

F. Deep Soil Improvement Techniques

As mentioned previously, the Navy contract was a performance contract in lieu of a specific method contract. The contract gave the contractor latitude in his design and method selection provided the final criteria (i.e., stated minimum density and/or test results) was achieved. The improvement techniques considered were Vibro-compaction, Vibro-replacement (stone-columns), Compaction Grouting (CG), and Dynamic Deep Compaction (DDC).

Vibro-compaction and Vibro-replacement techniques have been extensively discussed throughout the paper, however, compaction grouting and dynamic deep compaction have not been mentioned. Hussin (1987) briefly describes the two

as follows:

Compaction Grouting involves the injection of a low slump grout under high pressure to densify granular soils through displacement, and reinforces cohesive soils with the resulting grout column. The grout pipe is inserted into the ground to the bottom of the soils requiring treatment. The pipe is extracted as the Compaction Grout is pumped into the soil.

Dynamic Deep Compaction involves repeated dropping of a heavy weight from considerable heights. The technique is best suited for densifying granular soils. The weight is dropped on a grid pattern over the site in one or more passes.

G. Equipment and Material

The deep soil improvement for SWFLANT and Missile Motor Magazines were both awarded to the same contractor within three weeks of each other for a combined total of over $6 million. The large size and short duration of this project necessitated one of the highest concentrations of equipment and materials for a deep soil improvement ever used in the United States (Hussin, 1987).

Figure 30 shows a site plan of SWFLANT controlled area, with areas of deep soil improvement shaded.

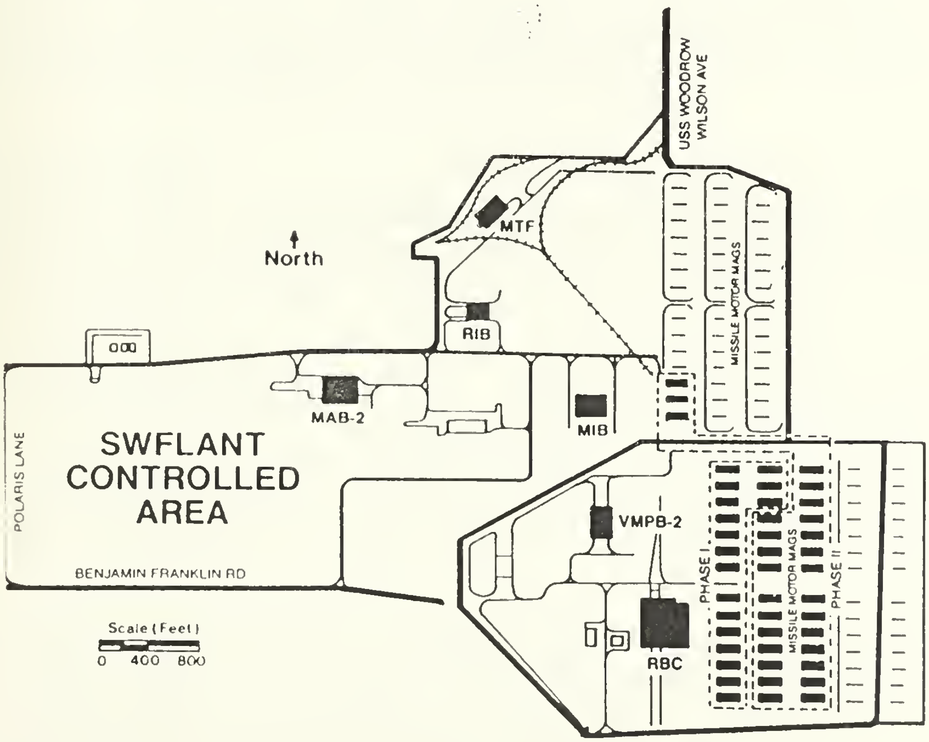

USS WOODROW WILSON AVE
MTF
North
RIB
000
MAB-2
MIB
MISSILE MOTOR MAGS
SWFLANT CONTROLLED AREA
POLARIS LANE
BENJAMIN FRANKLIN RD
VMPB-2
PHASE I
MISSILE MOTOR MAGS
PHASE II
RBC
Scale (Feet)
0 400 800

Figure 30. Site Plan of SWFLANT Area. (Hussin, 1987)

Table 2 is a summary of deep soil improvement information.

TABLE 2
SUMMARY OF DEEP SOIL IMPROVEMENT INFORMATION

Structure	Footprint Area		Improvement Depth		Number of Compaction Points	Treatment Method
	(sf)	(sm)	(ft)	(m)		
Reentry Body Complex (RBC)	174,200	16,192	48	14.6	2,100	VR
Vertical Missile Packaging Building-2 (VMPB-2)	43,300	4,025	43	13.1	487	VR
Missile Inspection Building (MIB)	49,900	4,638	42	12.8	604	VR
Motor Transfer Facility (MTF)	38,200	3,550	47	14.3	452	CG
Motor Assembly Building-2 (MAB-2)	56,500	5,252	45	13.7	690	DDC/CG
Radiographic Inspection Building (RIB)	65,600	6,098	46	14.0	51/49	DDC
Missile Motor Magazines Phase I	54,000	5,019	30	9.1	700	CG
	129,600	12,046	30	9.1	1,680	VR
Phase II	75,600	7,027	30	9.1	980	CG
	118,800	11,043	30	9.1	1,540	VR

The materials used for the project were sand, stone, and grout. The sand was obtained from local borrow pits relatively close to the project site. This insured compatibility with the in-situ material. The stone was a coarse granite ballast with a maximum dimension of 2 inches.

H. Techniques Considered

Although the overall site displayed similar characteristics, each individual site displayed subtle variations making different techniques considered attractive for different sites.

Missile Motor Magazines (MMM): The subsurface soil profile at the MMM was similar to the "generalized" profile. After preliminary design review, it was decided the area of stabilization was concentrated in the initial 30 feet. For the most part, the soils requiring treatment fell between depths of 13 to 25 feet, and consisted of silty fine sand to fine sand. Due to the proximity of adjacent completed structures, Dynamic Deep Compaction was not considered. For the initial testing phases, Vibro-compaction and Compaction Grouting were chosen.

After several tests using the large 165 Hp vibrator, it was discovered that a 3 to 5 foot lens of silty sand immediately under the hardpan could not be densified sufficiently. In order to significantly improve the performance of this layer, the backfill material was changed to stone and thus Vibro-replacement employed. The grid pattern chosen proved adequate with a 9 foot

square grid, and thus this area was successfully densified.

In addition to the Vibro-compaction techniques, Compaction Grouting was also used in selected locations. Areas with the thinnest silty sand lenses were chosen for this treatment. Compaction Grouting is most effective and economically attractive when treating individual zones and by-passing zones not requiring treatment (Hussin, 1987).

Radiographic Inspection Building (RIB) Site: Unlike the MMM, the subsurface profile at the RIB site varied from the "generalized" profile in two ways. The near surface "hardpan" layer was nearly as dense as elsewhere on the site, and the initial 30 feet requiring treatment consisted entirely of clean sands. These characteristics are ideal for both Vibro-compaction and Dynamic Deep Compaction, provided a large weight and significant drop height are used. For this site, Dynamic Deep Compaction was chosen. This gave the contractor a chance to test this procedure under ideal conditions. To achieve the required compaction, a 32 ton weight was dropped from a height of 100 feet. The drop grids (beyond the scope of this paper) were as follows: Primary drops were located on a 35 foot grid with as many as 30 drops per location. Secondary locations were at the center of the primary grid with as many as 15 drops per location.

The groundwater table at this location was at approximately 4 feet beneath the surface. Dewatering was necessary since the minimum ground water depth of 8 to 10 feet is required to permit the most effective use of DDC or Vibro-compaction (Hussin, 1987). A dewatering system was installed to lower and maintain the groundwater at 11 feet below grade.

Motor Transfer Facility (MTF) Site: The subsurface profile at the MTF site was nearly the opposite of the RIB site. The test results in the near surface soils satisfied the specification prior to treatment and the typical depth interval requiring treatment was from 13 to 16 feet and from 34 to 49 feet. Therefore, the soil requiring treatment was below a depth of 13 feet and in two or three distinct zones. These characteristics made the Compaction Grouting technique (CG) attractive. The procedure and spacing was similar to that outlined in the MMM section.

Missile Assembly Building 2 (MAB-2) Site: The subsurface profile at the MAB-2 site was similar to that at the MTF site except that some improvement in the near surface soils was required. Therefore, a limited Dynamic Compaction program was performed to densify the near surface loose soils, followed by a Compaction Grouting program to treat the deeper soils. DDC procedure and spacing were similar to that performed at the RIB site; however, only 20 drops at primary locations and 10 drops at secondary locations were required. The Compaction Grouting procedure and spacing was similar to that performed at the MMM sites.

Vertical Missile Assembly Building 2 (VMPB-2), Missile Inspection Building (MIB) and Re-entry Body Complex (RBC) sites: The subsurface profiles at the remaining sites were all very similar to that of the generalized soil profile. The profile indicated treatment was required between the depths of 13 to 43 feet. Included in the range were both cohesive and granular soils. Because of the mixture, Vibro-replacement was selected. A 165 Hp vibroflot was again used, with a stone backfill. This enabled the densification of the granular soils, as well as the reinforcement of the cohesive soils. The grid pattern chosen was triangular with 8 foot spacings.

I. Testing and Results

An extensive testing program was undertaken by the contractor and in cooperation with the Navy. Although not required by the contract, the contractor performed both pre-compaction and post-compaction testing. As stated earlier, the contractor was only required to perform post-treatment testing to prove the required results were achieved. Due to the magnitude of the site and the large build-up of soil improvement equipment, many individual companies/institutions requested permission to independently perform tests. Permission was granted on a not to interfere basis and thus a large amount of information should eventually become available.

To accomplish the testing program, a state of the art CPT/DMT truck performed testing a minimum of 40 hours per week with a second shift in the final weeks. The field testing consisted of approximately 13,200 linear feet of Electronic Cone Penetrometer Testing (CPT), 3,300 linear feet of Dilatometer Testing (DMT) and 250 linear feet Standard Penetration Testing (SPT). All field testing was performed at the mid-point of the treatment grid to test the loosest condition.

All post-treatment tests were performed within one week of soil improvement. Figure 31 presents test results for the vibrodensification sites. Since Dynamic Deep Compaction and Compaction Grouting were also performed of the site, Figure 32 presents the results of these techniques.

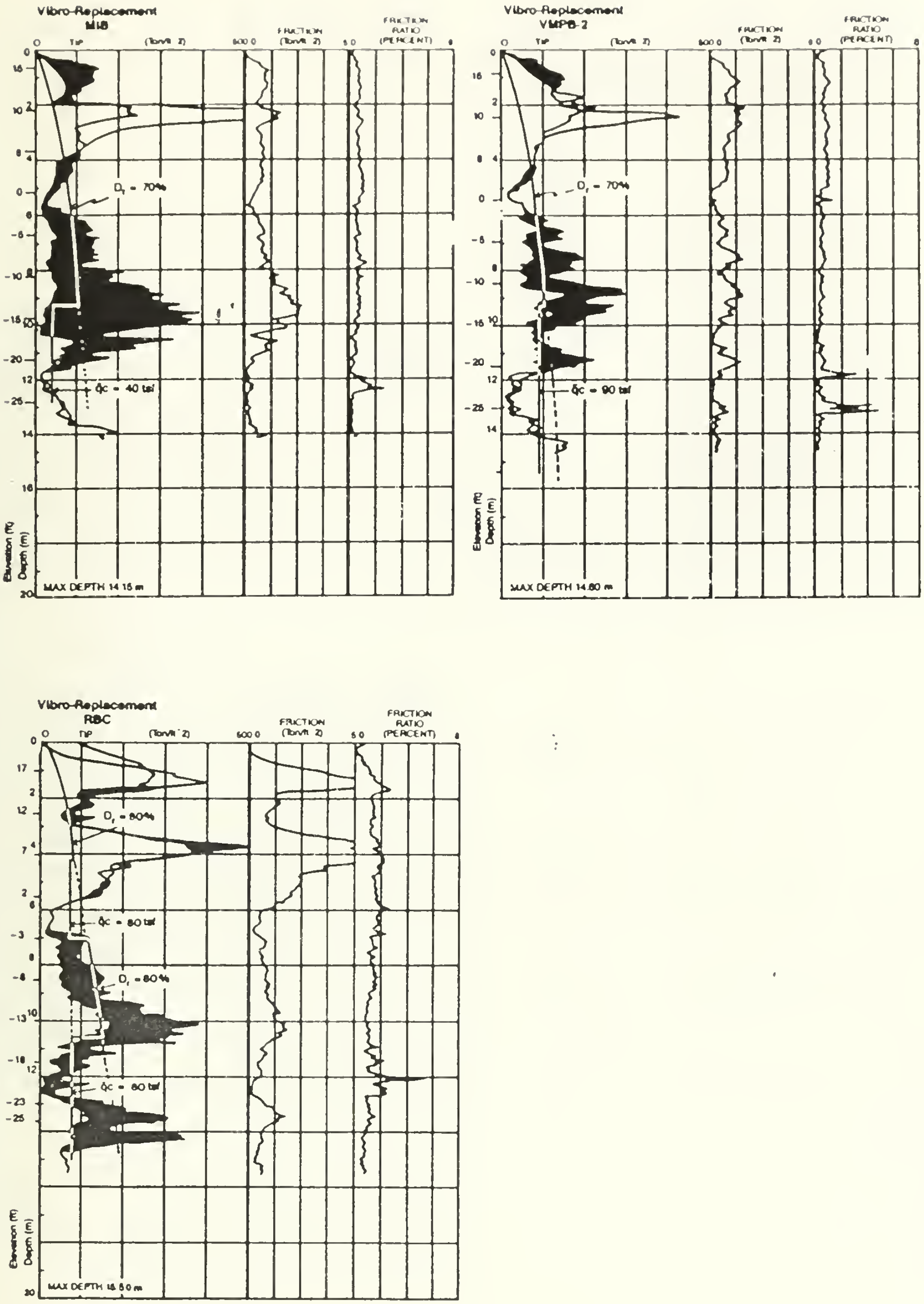


Figure 31. Sample CPT plots from sites treated by Vibro-replacement. Before and after tip resistance values are plotted with the improvement shaded. Specified improvement criteria shown on plots include both minimum relative density (D_r) and minimum CPT tip resistance (q_c). (Hussin, 1987)

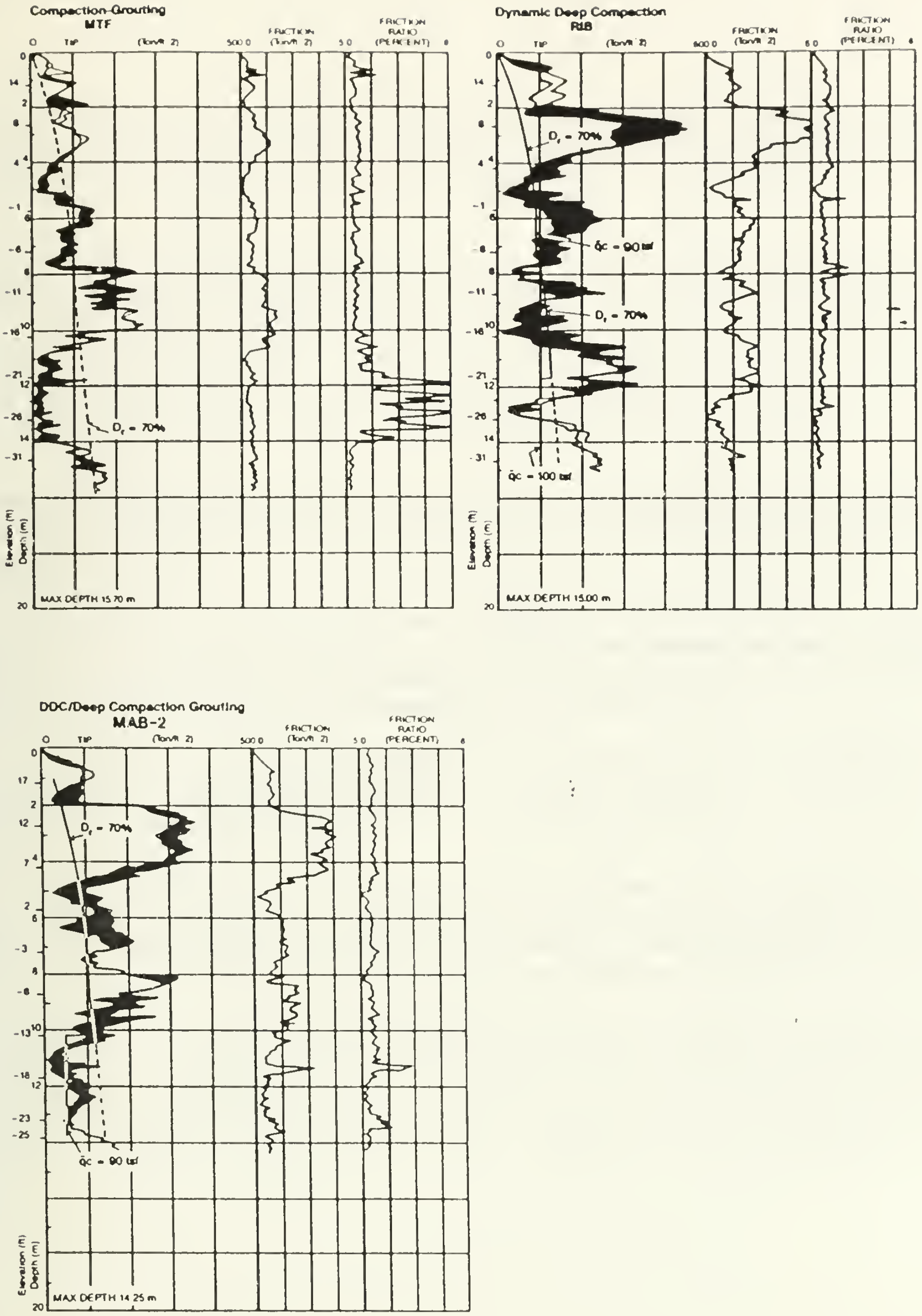


Figure 32. Sample CPT plots from sites treated by Compaction Grouting and/or Dynamic Deep Compaction. Before and after tip resistance values are plotted with the improvement shaded. Specified improvement criteria shown on plots include both minimum relative density (D_r) and minimum CPT tip resistance (q_c). (Hussin, 1987)

When viewing the figures, keep in mind the specifications requiring average test values over 5 foot depth intervals. Also, in soils with greater than 12% fines, the test value of the replacement material (i.e., stone or grout) was also included in the average with the natural soil.

Figure 33 presents the mean value of all pre- and post-treatment CPT values for each site with the values averaged over 5-foot depth increments. Also shown for each increment is the range of one standard deviation of the test results from the mean. Note in the MTF and MAB-2 results that Compaction Grouting was only performed in the depth intervals shown. Also, in the RBC, VMPB's, and MIB results, Vibro-replacement was only performed below the hardpan layer, with sand backfill above this depth.

Based on results achieved and from the data available both from Hussin (1987) and the Naval Facilities Engineering Command, the following observations are made:

Generally, by allowing the contractor to average into the test results the values of the fill material (i.e., stones or grout) a less conservative improvement is obtained. This must be considered in the final designs of the foundation system ultimately placed on the improved soil.

As expected, as the percentage of fines approached and surpassed 20%, the appreciable improvement between both Vibro-compaction centers and Vibro-replacement centers decreased.

A greater improvement of natural soil between stone column centers vs. Vibro-compaction centers was achieved. However, with the information available it is believed this was attributed to the test method allowed by the specifications, and not necessarily the performance of each technique.

Vibrodensification methods showed a greater improvement than Dynamic Deep Compaction in sands at depths greater than 25 feet. This can be attributed to the limited range of the 32 ton weight dropped from a height of 100 feet. If larger weights were used, a better correlation between the two techniques, theoretically, would have occurred at greater depths.

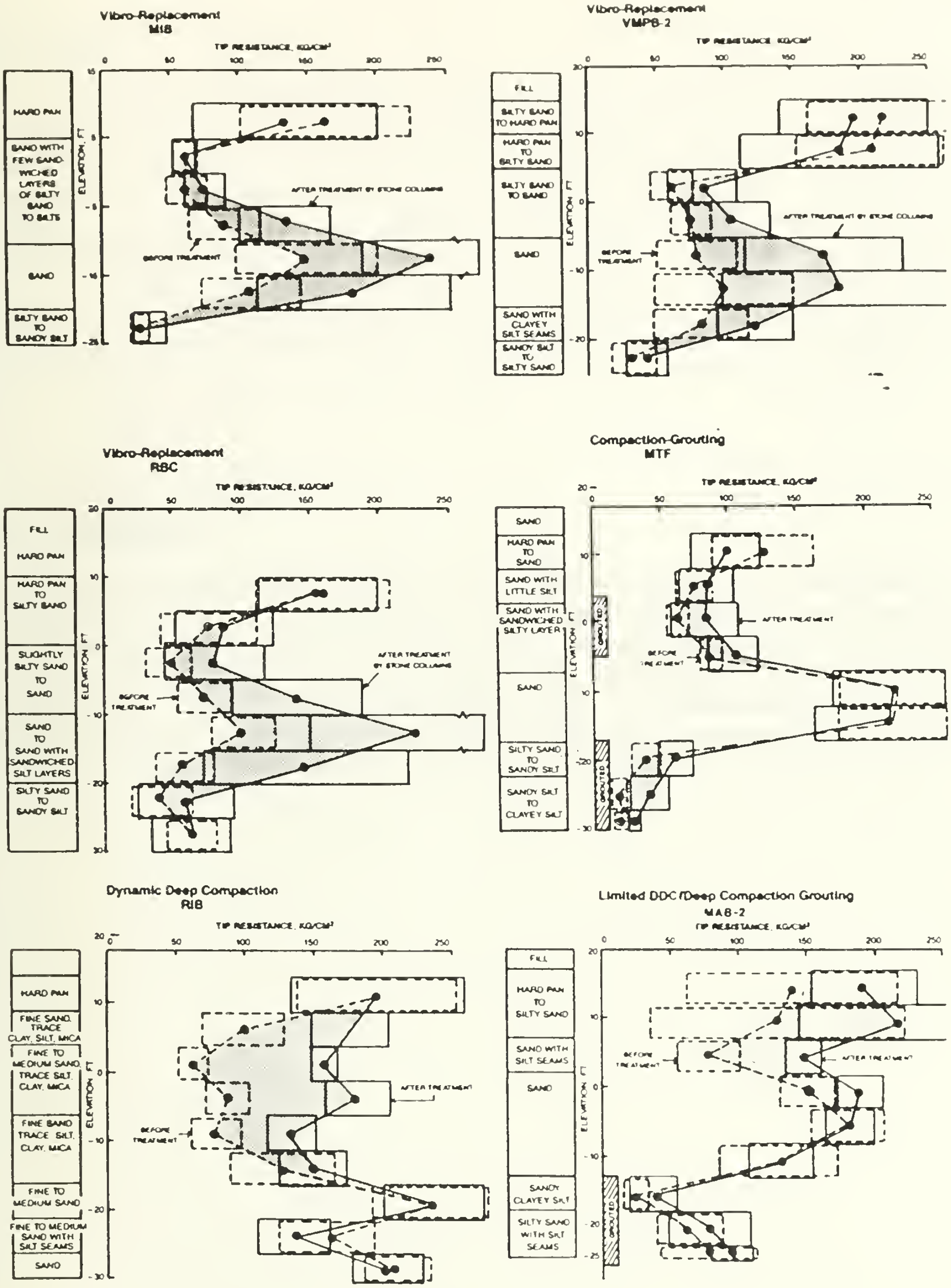


Figure 33. Plots of mean CPT tip resistance values for all before and after treatment tests at each site. Values are averaged over 5-foot depth increments with the improvement shaded. Also shown as boxes are the ranges of one standard deviation from the mean. (Hussin, 1987)

J. Conclusion

All densification techniques chosen were successful in achieving the improvement required. The clean sands were densified to the relative density required (65% to 70%). When cohesive soils were encountered, stone columns were constructed to reduce potential settlements. In addition to a successful soil improvement program, this project also shows the importance of a performance contract. The contract, as written allowed the contractor the latitude to choose the various methods available to achieve the required compaction. In addition, the methods chosen proved to be the most economical and efficient. This lowered the cost to the owner (i.e. U.S. Navy) and permitted the successful on-time completion of a difficult project.

K. Acknowledgment

The Naval Facilities Engineering Command (NAVFAC) in Alexandria, Virginia was extremely helpful in providing information on the Trident Base Project. The information was provided solely on an educational basis, and my position and rank within the U.S. Navy Civil Engineer Corps should not be mistaken as representing the views of the U.S. Navy. The information provided by NAVFAC, along with the paper published by James D. Hussin provided all information on the Case Study, and represents the authors views and opinions only.

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